HELENA city county SEWER & WATER

NORTHWEST PLANNERS

GENERAL PLAN

HELENA

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April 14, 1971

Honorable Mayor, City Commission, County Commission
and Planning Board
Civic Center Building
Helena, Montana 59601

Gentlemen:

We are pleased to submit herewith the technical report of the Helena City-County Sewer and Water General Plan in fulfillment of the requirements of our Contract with the City-County Planning Board.

We are indebted to many individuals and agencies who contributed time and information. With apology for possible omission, a list of acknowledgments is included.

Respectfully Submitted,

N. J. Campeau

N. J. Campeau

NJC:cmb

GRANITE BLOCK HELENA, MONTANA 59601 — PHONE 406-442-3930

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CHAPTER I

INTRODUCTION

The planning of water and sewer facilities in the past has often been conducted on a piecemeal basis with facilities designed to meet localized needs or crisis situations. The decision to conduct this study provided the opportunity to view water and sewer improvements within the entire system. By examining how each line contributes to the overall functioning of the system, problems have been isolated and corrective measures applied to put the system's facilities in proper balance with integral demands.

This general plan incorporates the results of a very thorough inventory and evaluation of Helena's existing water supply and distribution systems and waste water collection and processing facilities. It provides a plan for correcting existing deficiencies and meeting the demands imposed by future growth. A less comprehensive examination is made of the jurisdictional area requirements and of Helena's storm drainage system and requirements.

Priorities of recommended improvements within each system have been established based on apparent need or magnitude of existing problem. The priority of water improvement needs as against sewer improvement needs must be based on a consensus of personal opinions or by environmental health requirements.

This report is divided into four distinct parts. The first part discusses the present and future population, its distribution and future impact areas that the present and future utility systems must be designed to serve. The second analyzes the existing water system and recommends a future system that will serve 1980 and 1990 demands. The third part of the report covers existing and future sanitary sewer needs with recommendations and alternatives, while the fourth and final section deals with the storm sewer system serving the community.

CHAPTER II

EXISTING AND FUTURE DEMANDS

Theoretically, the municipal demand for water can be measured by metering water entering the system from all supply sources. The sanitary sewage contribution can also be measured by metering flow through the sewage treatment plant. This assumes, of course, there is no leakage in the water supply or distribution systems and that there is no entry of ground water or storm drainage into the sanitary sewer system.

To determine future demands, it is necessary to relate existing demands to existing conditions and then to project future conditions that will have an effect on the utility systems.

The two most important factors that influence sewer and water demands are population and land use. These two variables were used to arrive at future water and sewer demands in this study. Water consumption and sewage discharge can be related directly to the number of people occupying a residential area. While population in a residential area can be accurately estimated, the consumption and discharge of water and sewer for commercial, industrial and public uses varies considerably. Thus, the use of land within the study area was determined and the relationship between non-residential floor space and existing water and sewer demands established. Once these relationships were established, future water and sewer demands were estimated by projecting future population and land use.

The Montana Highway Commission is presently preparing a Transportation Plan for the Helena Jurisdictional Area which has been divided into 119 traffic zones for analysis purposes. Population estimated have been made for each of these zones. These same analysis zones were utilized to estimate present and future loadings on the water system. In some cases it was desirable to combine several analysis zones to correspond with the skeletonized water supply system, however, no adjustment to the population distribution was necessary.

The sewer collection system does not serve these same analysis areas. Because this system relies upon gravity flow to transport the wastes to the treatment plant, it was necessary to establish separate analysis zones or collection areas and to estimate the flows into the system from each area.

POPULATION FORECASTS

Population projections prepared by the City-County Planning staff were the basis for determining population densities and the distribution of population for each water system analysis zone. Although the estimated total city population for 1970 was 14.55 percent over the actual Census count, it was assumed that the relative distribution of the population was valid. In order to adjust the estimated population with the Census figure of 22,730 persons for the City of Helena, a 14.55 percent reduction factor was applied in each traffic analysis zone. The same adjustment was applied to the 1975, 1980, 1985 and 1990 population projections. For the area outside the city limits the City-County Planning Board figures were used with no adjustment.

A 1968 land use inventory by the City-County Planning Board tabulated commercial, industrial and public use floor area for each analysis zone. This data was used to relate zonal water and sewage demands for non-residential uses.

WATER CONSUMPTION

Studies of the water supply system have assumed per capita consumption rates ranging from 220 gallons per capita per day,[1] to 240 gallons per capita per day.[2] The most recent estimate available is 220 G.P.C.D. from the 1955 report.[1]

Figure 1.1 shows the yearly water consumption from City of Helena water sources for the years 1960 through 1970. Based on these amounts, the average daily per capita consumption of water in 1960 was actually 256 G.P.C.D., while 1970 daily per capita consumption averaged 313 G.P.C.D.

If water use were projected on a straight-line basis over the planning period, the average Helena resident would be using 370 gallons per day in 1980 and 427 gallons per day in 1990. This was considered unrealistic from the standpoint

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[1] A Report Covering Potential Source of Water Supply for a Long Range Plan of Development of the Helena Water System, November 1955, Phillips-Carter-Osborn, Inc., Denver, Colorado.

[2] 1950 Report, Morrison-Maierle, Inc., Helena, Montana.

of economics as well as probable future needs. Water rate increases would be needed to meet these levels of consumption. Such increases would reduce the rate of consumption.

An average daily per capita rate of consumption of 320 gallons was assumed for the entire 20 year planning period.

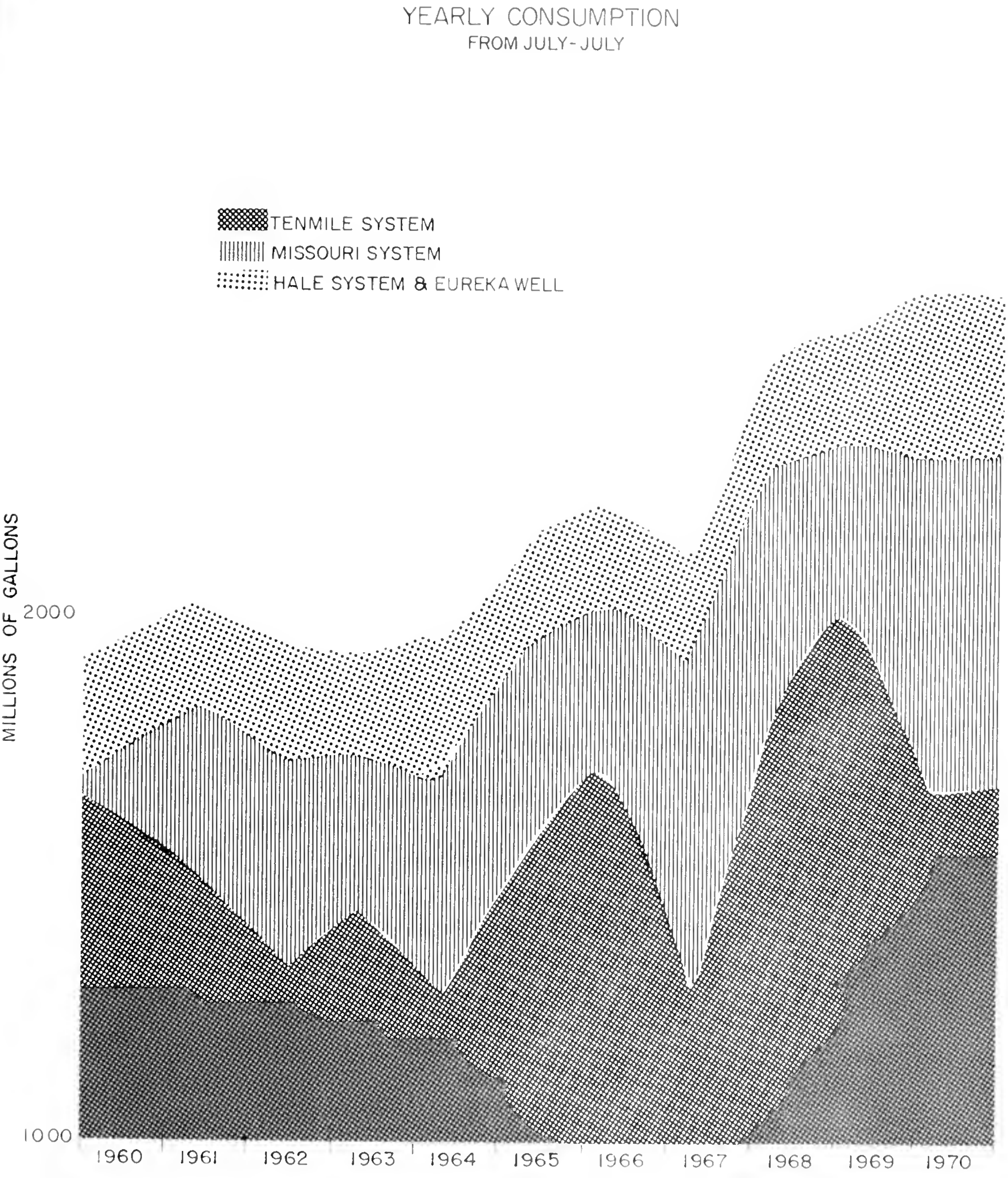
The per capita rate within the city limits was calculated by dividing the total water consumed in 1970 by the 1970 Bureau of Census population. It was then necessary to distribute this quantity of water to the areas of consumption so that the water distribution system could be evaluated. On the basis of the average distribution of water use in other cities, the following breakdown for each land use was estimated as shown in Table 2.1.[3]

Table 2.1: Average Water Consumption by Land Use.

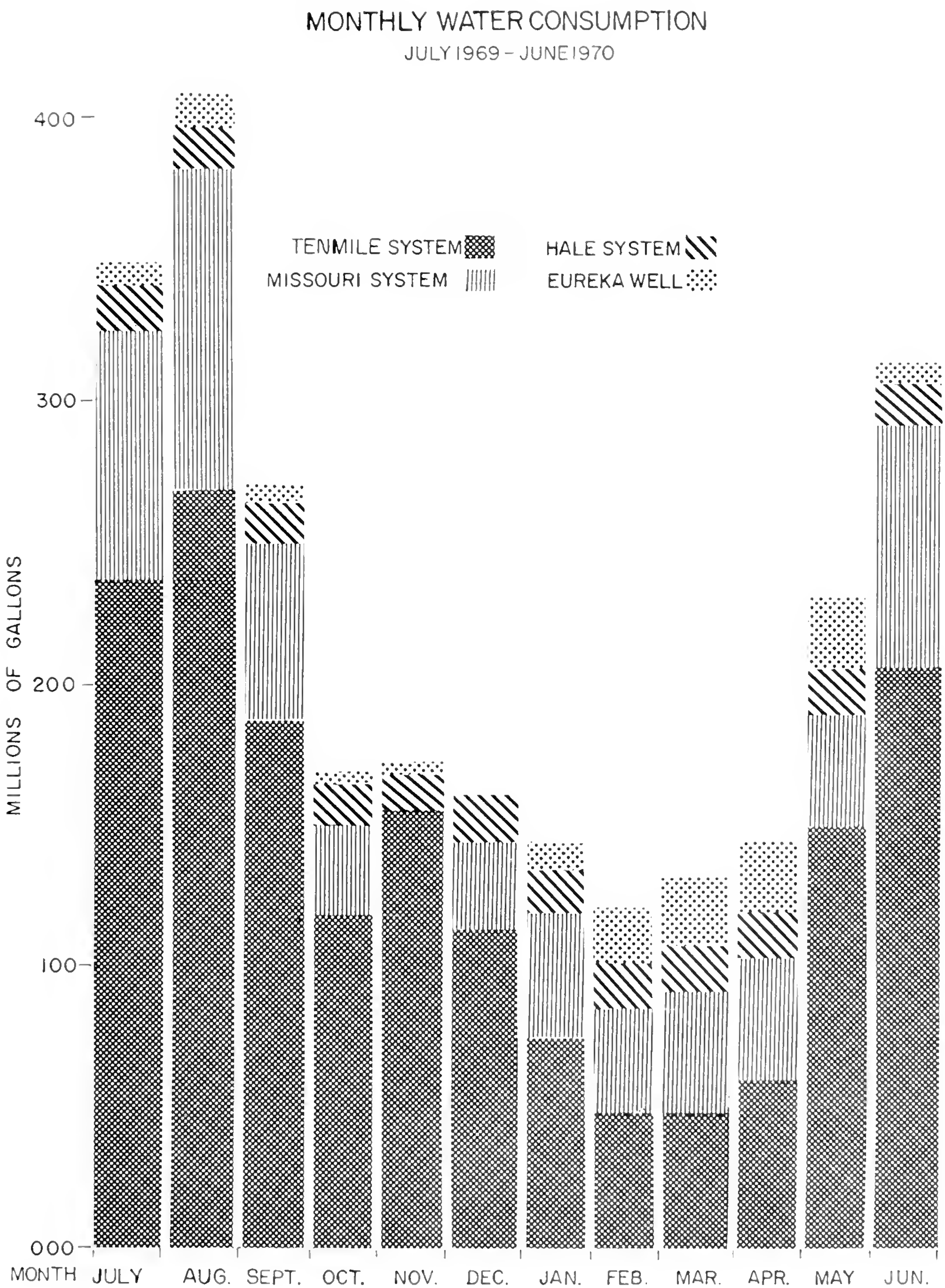
Land Use		Gallons/Capita/Day
Residential		210
Commercial		24
Industrial		61
Public		13
Water Unaccounted for		12
	Total	320

It was then necessary to allocate the amount of water consumed for each land use to geographic locations. Residential consumption was assigned to each analysis zone based on the number of people estimated to be living in each zone at present and in the years 1975, 1985 and 1990. Commercial, industrial and public water demands were based upon the number of square feet of each of these uses in each analysis zone as

[3]Average total consumption was taken from city water records. Commercial, industrial, public and unaccounted water consumption based on percentages from "Elements of Water Supply & Waste Water Disposal", Sept. 1961, p. 24, Gordon Maskew Fair and John Charles Geyer, John Wiley & Sons, New York and London.



YEARLY CONSUMPTION
FROM JULY-JULY
TENMILE SYSTEM
MISSOURI SYSTEM
HALE SYSTEM & EUREKA WELL
MILLIONS OF GALLONS
2000
1000
1960
1961
1962
1963
1964
1965
1966
1967
1968
1969
1970


MONTHLY WATER CONSUMPTION
JULY 1969 - JUNE 1970
TENMILE SYSTEM
MISSOURI SYSTEM
HALE SYSTEM
EUREKA WELL
MILLIONS OF GALLONS
400
300
200
100
000
MONTH
JULY
AUG.
SEPT.
OCT.
NOV.
DEC.
JAN.
FEB.
MAR.
APR.
MAY
JUN.

determined in the 1968 land use survey.[4] Future commercial, industrial and public demands were allocated to each traffic zone in proportion to the amount of land zoned for that specific use. Finally, water unaccounted for was distributed to the analysis zone in proportion to the water demand for all other uses. In most cases,water unaccounted for is lost from the system due to leakage or unmetered uses.

One final type of water demand was then added to selected analysis zone. This was the demand which would result from specific future developments which are planned for the jurisdictional area but which were not recognized in the population estimates and distribution or in the floor space inventory. These impacts were determined from interviews with the prospective developers and the City-County Planning Staff. Water demands for these impacts were estimated and added to the calculated demand for the appropriate analysis zone.

The final product of these various demand estimates was a composite estimated average daily water demand for each analysis zone for present use and future development. These projected demands were then superimposed onto the existing water system through a computer program to determine present and future deficiencies. This computer analysis is discussed in a subsequent section of this report.

The sanitary sewage flow comes from several sources including the spent water of the community, ground water seepage and stormwater runoff. These three sources were considered in estimating sewer flows for each analysis zone. Water consumption was used as an index to estimated the amount of spent water entering the sewage collection system. The method of estimating these flows is covered in detail in the chapter on the Sewer System.

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[4]Land Use Survey - Consulting Services Corporation.

CHAPTER III

EXISTING WATER SUPPLY SYSTEMS

The oldest water systems in Helena were originally developed to supply the placer mining operations during the gold rush of the 1880's. After the placer workings were depleted, these systems were utilized for municipal water supply.

Among the oldest supply systems was the Woolston system which consisted of a large diameter shallow well and pumping plant in the vicinity of the fairgrounds and the present Woolston Reservoir. Other old systems include the Bed Rock and Eureka systems which take ground water from the bedrock level of Last Chance Gulch by well and pump, and the Hale, a system which takes water from a higher tributary of Last Chance Gulch.

The early distribution systems utilized a thin walled wrought iron pipe with the trade name Kolimain, which found widespread use in early western towns before the turn of the century because of its light weight and low shipping cost. Although appropriate for its time, the pipe is a source of trouble today since its thin walls deteriorate and fittings for connections are no longer available.

DESCRIPTION OF EXISTING SYSTEMS

The City of Helena obtains water for domestic and irrigation needs from three basic supply systems. These systems include the Ten Mile System, the Hale system and the Missouri River System. The accompanying map (Figure 3.1) identifies these systems and shows the basic water works facilities constituting each system.

Ten Mile System

The Ten Mile System was put into service in the early 1900's and has provided most of the municipal water supply for Helena since that time. In general, the Ten Mile water system services the upper and lower west sides of Helena. However, Helena's pipeline network is interconnected from the far west side of town to the far east side of town, thereby interconnecting the Ten Mile System service zone with the Missouri River system service zone. Existing city

water use records show that all of Helena, excluding that serviced by the Hale System, has been fed during wintertime months by Ten Mile, during which time the Missouri System was shut down. The capacity of Ten Mile System is nine million gallons per day.[5]

The Ten Mile System collects water from tributaries in the mountainous upper reaches of the Ten Mile drainage area. Surface water from snow-melt and rain constitute the major source of supply. This surface water may be supplemented in a very minor way by water pumped from two shallow wells in the Ten Mile subsurface alluvium.

The surface water is collected during the period of high spring run-off from two drainage areas adjacent to the Continental Divide and stored in the Scott and Chessman Reservoirs. The capacity of Chessman Reservoir is 550 million gallons[6] with a contributing watershed area of 6.7 square miles. Most of Chessman's watershed lies in the Banner Creek drainage which is not a natural tributary but which by diversion and the Red Mountain Canal becomes tributary to Chessman Reservoir.

The Red Mountain Canal is an aqueduct of approximately five miles in length consisting of flumed sections and sections of unlined, open ditch. Flumes are of galvanized sheet metal. Some of the flumed sections are carried on untreated timber trestles and some are set directly on grade (lined canal). There are also two minor sections which are piped. The following lengths of each type construction were obtained by field measurements:

Flume on trestle	-	4,880 feet
Flume on grade	-	6,890 feet
Open ditch	-	12,950 feet
Pipeline	-	480 feet

Each spring after the last snowfall city crews remove snow and ice from the Canal so that it can be in service when snowmelt begins. It is in use until Chessman Reservoir is full or until the enforcement of water rights forbids its

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[5]Per City Engineer, Robert Newhouse.

[6]City Water Billing and Collections Department records.

continued use. The water rights restriction was imposed to maintain flow in Ten Mile Creek after judication proceedings in 1964-1965. Red Mountain Canal is maintained by a ditch-walker who makes minor repairs, removes debris and diverts water as necessary to keep the canal full but not over-flowing. The capacity of this facility is ten million gallons per day.[7] Both Red Mountain Canal and Chessman Reservoir were constructed in 1908 as part of the Initial Ten Mile System.

Helena's second mountain storage facility, Scott Reservoir, was constructed in 1952 by increasing the height of the old Gould Reservoir which had been used for placer mining purposes. It was constructed to obtain supplemental supply so Chessman water storage could be conserved for wintertime use. Its capacity is 194 million gallons[8], and its drainage area 1.9 square miles.[7]

Both Scott and Chessman Reservoirs are connected to the Ten Mile System by manually operated valves which release water into the natural channels of Ruby and Beaver Creeks. These creeks converge near Rimini where the water is diverted into an eighteen inch concrete pipeline with a capacity of 8.9 million gallons per day.[9] Water is diverted into the concrete pipeline at downstream points at a number of diversions which collect water directly from streams near their confluences with Ten Mile Creek. Such diversions and intakes collect water from Minnehaha Creek and Moose Creek as well as directly from Ten Mile Creek at Rimini.

The eighteen inch concrete pipe follows the course of Ten Mile Creek for a little over seven miles to the Ten Mile Settling Basin which has a six million gallon capacity.[7] Ten Mile Settling Basin serves as a storage and equalizing reservoir as well as a basin where some clarification of

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[7] Helena Water System, Filter Plant Dedication by Planning Board, 1960.

[8] City of Helena Water Billing and Collections Department records.

[9] Equal to 550 miners inches - Decreed Water Rights Records, State of Montana Department of Water Resources.

turbidities occurs. It is here, below the effluent structure, that basic treatment of the raw water is performed by means of coarse screening and chlorination. The purified water is then metered and a continuous graph recorder in the chlorine house keeps a permanent record of flow rates and daily totals of water supplied by this source. The rate of flow of water to Helena is controlled by a manually operated valve near the Ten Mile chlorine house.

Water leaves Ten Mile Settling Basin by a 24-inch diameter steel pipeline and two 16-inch cast iron pipelines. Each is about 5-1/4 miles in length. According to the City Water Department, the 16-inch cast iron transmission mains were in service for many years before the 24-inch steel main was added in 1948.[10] These steel and cast iron pipelines extend from Ten Mile Settling Basin down Ten Mile Creek to a terminus at Yaw Yaw, which is about a mile west of the State Nursery greenhouses. Records show an interconnection of the pipelines at Yaw Yaw.[11]

From Yaw Yaw there are two wood stave transmission mains which bear eastward up a steep hillside and a cast iron main which continues northward following U.S. Highway #12 and then turns eastward following Cannon Street. (See Fig. 3.1) The wood stave mains are of 16-inch and 24-inch diameters, and the cast iron main is 16-inch reducing to 14-inch for some distance and finally reducing to 12-inch diameter on Cannon Street.

The 16-inch and 24-inch diameter wood stave transmission mains are of differing ages and construction. The 16-inch main is older and has probably been in service since around the turn of the century. It is reported to be redwood stave and was laid using prefabricated, metal, spirally-banded sections. These sections were connected with the use of wooden collars which were tightened by steel bands.

The 24-inch wood stave transmission main was constructed in 1941.[10] This pipe was fabricated in place with city forces

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[10] Per Mr. George Malben.

[11] Plans of 24-inch main - City of Helena Engineer's Office.

and by people working in payment for delinquent water bills. The W.P.A. also furnished manpower in the construction of an 800 foot section which had to be tunneled.[12] The staves were tongue and groove pieces about 3-1/2 inches wide and banded together at intervals with steel tightening bands to give a circular pipe of about 1-1/2 inch wall thickness. Apparently there was no protective coating applied to the outside of the pipes. The wood appears to be West Coast cedar.

These wooden transmission mains extend from Yaw Yaw to the Woolston Reservoirs which are located on the east side of Mount Helena, west of Last Chance Gulch. Water from the mains may feed the two reservoirs that have 6.18 million gallons combined capacity or goes directly into the distribution system through a 16-inch main on Holter and two 12-inch lines from the Reservoirs. (See Fig. 2.8) The reservoirs "float" on the system, for the only method of controlling their water level is by looking into them and manually adjusting the rate of flow in the transmission mains by turning the gate valve at Ten Mile Settling Basin.

The Woolston Reservoirs furnish the largest part of Helena's reserve storage which is necessary for fire protection and to meet peak hourly demands. In recent years these reservoirs have been nearly depleted during July and August, the months of peak demand. This year, 1970, water rationing was necessary on the west side of Helena.

Hale System

The Hale System provides only a minor portion of Helena's water supply. This system obtains water from two sources; Oro Fino Gulch and the Eureka well. Water from Oro Fino Gulch, south of the city, is diverted into a ten inch pipeline and flows by gravity for about 3-1/2 miles into Hale Reservoir. City records indicate that this pipeline normally flows continuously at a rate of about one-half million gallons per day.[13] During periods of high demand, this source may be supplemented by pumping water from the Eureka well through an eight inch pipeline into Hale Reservoir. The Eureka well is

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[12] Per Mr. George Malben.

[13] City of Helena Water Billing and Collections Department records.

located on Last Chance Gulch near the Old Brewery Theatre. This well and pump installation can also pump water directly into the major Helena distribution system. The maximum capacity of the Eureka well, based on monthly records[14], is small; about one-half million gallons per day.

The Hale Reservoir furnishes reserve storage for the Hale System of 2.29 million gallons. A small booster pump obtains water from the Hale Reservoir and pumps it to an 18,000 gallon tank located on Rodney Street to supply a limited number of homes in the area which are at a higher elevation than Hale Reservoir. (See Fig. 3.1)

Hale System's present service zone is shown on Figure 3.8. It is completely isolated by closed valves from Helena's major pipe system and therefore operates as a separate distribution system. In the past, its service zone was larger but it has been valved off at higher elevations and made smaller as the upper areas have developed. Its service limits reach to an elevation of about 4,280 feet, which is somewhat higher than the Ten Mile and Missouri service zones. These go approximately to elevation 4,200 feet.

<u>Missouri River System</u>
On the recommendation of a 1955 study conducted by Phillips-Carter and Osborn, Inc., the City of Helena participated in the Helena Valley Project for a fifty year water supply by contracting with the United States Bureau of Reclamation for water rights of 5,680 acre feet per annum. This is equivalent to slightly over five million gallons per day for 365 days per year. Two years later plans were completed for construction of facilities rated to supply treated water to the city at a maximum rate of six million gallons a day.

These facilities included a raw water supply line, a water treatment plant and a treated water supply line. The four million gallon Malben Reservoir, a part of this system, was already in existence. The several elements of this system are indicated on Figure 3.1.

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[14] City of Helena Water Billing and Collections Department records.

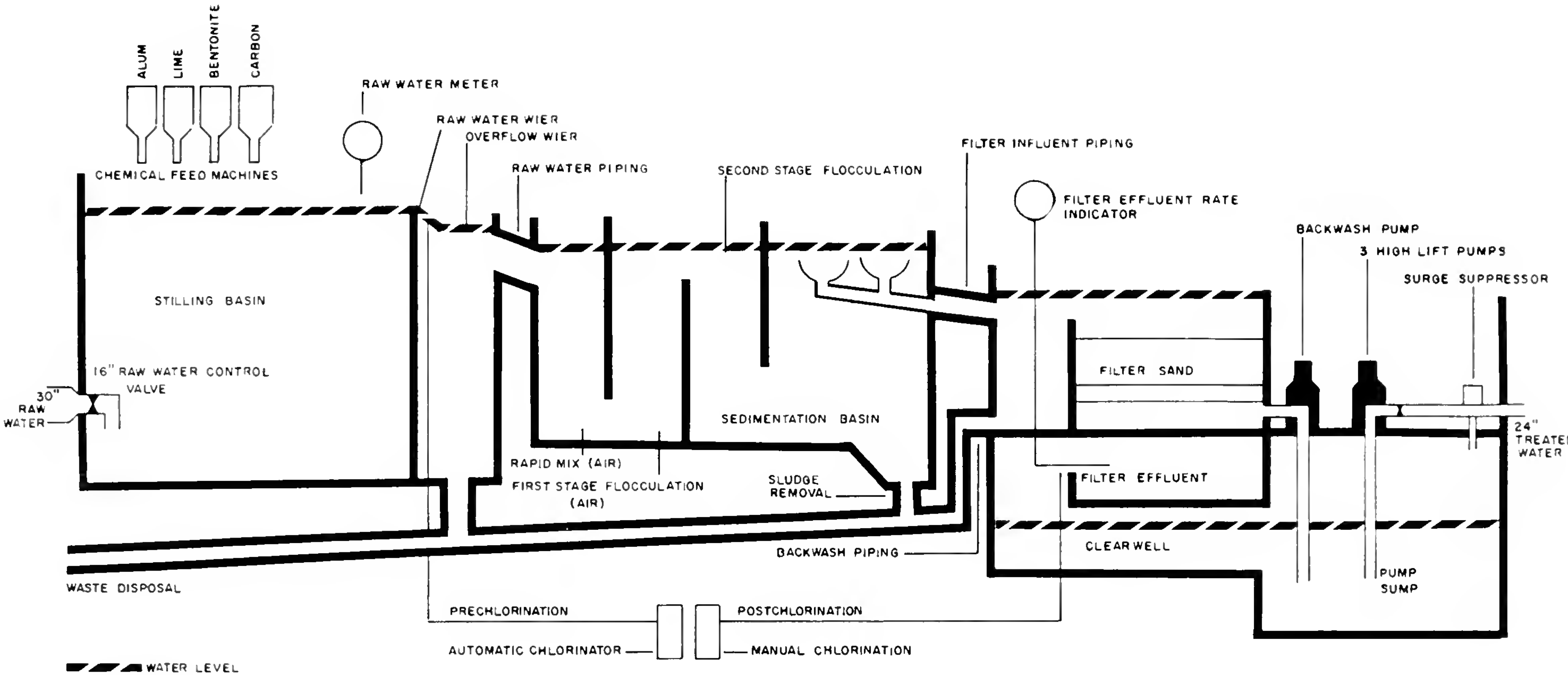
Raw water from the Missouri River is delivered to the Bureau of Reclamation equalizing reservoir, located midway between Helena and Canyon Ferry, by a tunnel and canal constructed and maintained by the U.S. Bureau of Reclamation. The City of Helena purchases the water from the Bureau of Reclamation at the reservoir and delivers it to the Helena system through city owned and maintained supply lines.

That portion of the system operated by the City of Helena begins with an intake structure at the dam of the equalizing reservoir. From this point, a 30-inch steel pipeline carries raw water by gravity to the treatment plant 3.9 miles away. The transmission pipeline was designed for future treatment plant expansion. Specifications call for a design capability of 12 million gallons per day. The rate of flow is controlled by a 16-inch pneumatic butterfly valve which normally operates automatically to adjust plant influent rate to equal the rate of flow passing the filter effluent wier.

The Helena water treatment plant is a conventional rapid-sand filter design. Figure 3.2 shows a schematic of the plant. Chemical pretreatment is performed as the water enters a stilling basin. Chemicals normally added are alum, lime and bentonite, although carbon may also be added to control tastes and odors. The raw water from the stilling basin is metered and prechlorinated over a cipolletti wier and is piped into two rapid-mix basins.

It then flows downward through the rapid-mix chambers and through holes into two flocculation basins. Compressed air diffused at the bottom of the basins is used in rapid-mixing and first stage flocculation. From first stage flocculation, the water flows through holes near the top of the basins and downward through four "second stage" flocculation basins.

The water then flows under partition walls and upward and then enters the sedimentation basins. The four sedimentation basins each contain two collection troughs which convey water into the filter influent piping. This piping carries water from the sedimentation basins and dispenses it to eight sand filters, each of which has an operating console. The filters consist of 24 inches of filter sand on the top and 12 inches of gravel on the bottom. A commercial underdrain system collects filter effluent.



HELENA WATER TREATMENT PLANT - SCHEMATIC FIGURE 2.2

Rate-of-flow controllers for each filter may adjust the rate of filtering from 0.3 - 1.125 M.G.C. (million gallons/day). The filtered water flows into a filter effluent well of 30,500 gallon capacity and then overflows across a five foot wier where the rate of flow is monitored and postchlorination is performed. After passing the filter effluent wier the treated water enters a clearwell of 180,000 gallons capacity. Part of the clearwell is sumped for pumping.

High lift pumping is accomplished with three 400 horsepower multistage pumps each rated at 2,000 gallons per minute against a 680 foot head of water. Only two of these are operated at once to supply six million gallons per day, leaving one for standby. Backwashing is done by a fourth pump but may be performed by the highlift pumps if necessary, using a valve and pipe bypass arrangement. A surge supressor is teed into the high service discharge manifold and is for the purpose of relieving the surge in the treated water supply line that results from pump shutdown.[15]

Backwashing is accomplished by pumping water from the plant clearwell back through the filter underdrains and forcing it up through the sand filters. This washwater is collected in troughs above the filters which convey it into an 18-inch waste drain pipe. This pipe carries backwash water and sludge from the sed-basins into an overflow sump. From here waste is carried by gravity pipe and ditch into nearby Prickly Pear Creek.

Treated water leaving the high service pumps is discharged into a 24-inch steel pipeline which extends towards Helena for 1-1/2 miles and then separates into two 20-inch mains. One of these mains extends for about a mile to the west and connects to a six inch pipeline in the vicinity of Helena Airport. This six inch pipeline extends under Interstate 15 and goes to services on the lower north side of the city. The other 20 inch main crosses the interstate highway at the Capital Interchange, extends up California Street to Winne, and then extends west on Winne to a point just west of Montana Street.

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[15]Contract Documents (Schedule 2-Water Treatment Plant, Sec. IV-J).

Some water may go to service mains tapped into the 20-inch pipeline on Helena's east side. The remainder of the water pumped goes to storage into the four million gallon Malben Reservoir near Montana Street (low demand condition) or goes into the distribution system (high demand condition) through a 12-inch service main on Dakota. (See Figure 3.8 for details of piping.)

Malben Reservoir is equipped with a water level transmitter, receiver and indicator at the treatment plant to allow the plant operator to monitor the level of the water in the reservoir. Records of this water level are also made by a continuous graph recorder at the plant. These records indicate that during hours of greatest demand the high service pumps do not maintain the water level in Malben Reservoir, meaning that reserve storage is being used to meet the peak hourly demands. During hours of lower demand the pumps gain on the reservoir. When the reservoir becomes full, the pumps and water treatment plant must be shut down.

The Helena water treatment plant was first put into operation in 1960 and has become a primary element in helping supply Helena's water. It is essential in meeting the high daily demand imposed by lawn irrigation during the summer months which has exceeded the capacity of the Ten Mile transmission system. Winter use records indicate that it has also been essential in supplementing the annual supply available from Ten Mile. The plant has normally been taken out of operation for part of the winter but as Chessman Reservoir approaches depletion before new runoff begins, the plant has had to be put back into operation. The period of plant shutdown has varied with the climatic conditions on Ten Mile watershed. During the period from July, 1968, to June, 1969, the plant was shut down for six months from October through March, while last year, 1969-1970, the plant was out of operation for only one month.[16] The trend has been towards shorter periods of wintertime shutdown as Helena's population has increased and the total annual consumption of water has increased. If the present trend increases, and as Helena's east side continues to develop, the water treatment plant will undoubtedly have to be operated all months of the year.

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[16]City of Helena Water Billing and Collections Department records.

ECONOMIC ANALYSIS OF PRESENT WATER SOURCES

In analyzing any municipal water supply, the economics of each source and the costs associated with supplying potable water to the distribution system at adequate pressures must be studied prior to developing a general plan. As part of this report the present unit costs for water supplied by each of the two major systems, Ten Mile and Missouri River systems, have been estimated.

Two types of costs are associated with any supply source; fixed annual costs and variable costs. Fixed costs are those which are independent of actual water use and include initial capital costs plus interest. Variable costs are those which vary proportionally with the units of use. Included in variable costs may be those of raw water, chemicals for treatment, power and operating costs.

In prorating fixed costs to unit costs, a life expectancy must be assumed for a facility, and the facility must be assumed to be operated at its practical capacity. For purposes of this report, a fifty year life was chosen for all stationary facilities and a twenty-five year life was used for mechanical equipment.

Cost information was taken from documents of the City Finance and City Engineer's offices. In analyzing sources, no costs have been included which would be associated with in-city distribution or maintenance of services. Maintenance of supply facilities was included in estimated operating costs and was assumed to be mostly in the form of labor.

In discussing the Missouri River source it must be noted that the City of Helena must purchase a minimum of 600 acre-feet of water annually at a cost of $16 per acre-foot according to the contract with the U.S. Bureau of Reclamation. For additional water up to the limits of their water rights (5,680 acre-feet) the cost is $14 per acre-foot. To be consistent with the water sources map, unit costs have been based on a "per million gallon" basis.

The following costs were computed for the Missouri River source operated at an estimated practical capacity of 5,680 acre-feet per year or 1,850 million gallons per year.

Unit Costs - Missouri System

Variable Costs (Based on '68-'69 records)

Raw water cost	$ 43.63/M.G.
Chemicals - Line, Alum, Bentonite, Chlorine	14.16
Power Costs	39.56
Operating Cost (Labor)	10.00
Miscellaneous (Gas, etc.)	3.89
Total Variable Cost	$111.24/M.G.

Fixed Costs = (interest @ 2-3/4% - 18 years)

Annual Cost - $53,063 ÷ 1850 = 28.68/M.G.

Total Unit Cost at Capacity $139.92/M.G.

It should be noted that the prorated unit fixed cost increases as the use of a source decreases. For instance, using the actual water supplied by the Missouri System during fiscal 1969-1970 $624.89 M.G. the annual fixed cost prorated to unit fixed cost becomes $53,063 ÷ 624.89 = $84.92/M.G. This added to the variable unit cost of $111.24 gives a total unit cost of $196.18 per million gallons supplied at last year's use rate.

In estimating present unit costs for water from the Ten Mile System, the capacity must be limited to annual available supply. For this study, the ten year's supply from Ten Mile (1960-1970) as recorded in the City Water Billing and Collections Office, averaged 1,558 M.G./annum. Initial capital costs were estimated from values contained in the 1969-70 auditor's report prepared by Galusha, Higgins, and Galusha. For comparative purposes interest rates and periods were assumed to be the same as used in estimating interests costs for the Missouri System.

Variable costs appear to be mostly in the form of operating expenses including salaries, vehicle expenses and furnishing housing for employees. Three regular employees were included at monthly salaries of $546, $483 and $441.[17] Costs of

[17]City Water Billing and Collection Department recorded figures.

overhead, vehicle and housing was included for each. Also included was the estimated cost of removing snow from and maintaining Red Mountain Canal during its months of operation.

The following schedule gives present unit costs estimated for water from the Ten Mile System.

Unit Costs - Ten Mile System

Variable Costs:

Operating and Maintenance	\$24.41/M.G.
Chemicals (Chlorine only)	1.00/M.G.
Misc. (Power, Gas, etc.)	1.00/M.G.
Total Variable Cost	\$26.41/M.G.

Fixed Costs:

Est. Annual Cost - \$72,277 ÷ 1558 = \$46.39/M.G.

Total Unit Cost at Capacity \$72.80/M.G.

From these estimated costs, it appears that water from the Ten Mile gravity system has been the more economical source of available supply for the City of Helena. Unfortunately, a number of the facilities have deteriorated and are in need of major repair or replacement. This matter is treated in another section of this report.

EXISTING WATER QUALITY

In addition to examining the economics of municipal water supply sources, it is essential to consider the quality of water furnished by each supply system.

The chemical properties of water from each of Helena's supply systems is based on records furnished by the Montana Board of Health. Their records are for water samples taken in 1960.

Ten Mile System water shows a hardness of 69 ppm as compared to 196 ppm for Missouri River and 156 ppm for the Hale System. This indicates that Ten Mile water has excellent qualities of softness while Missouri and Hale water is somewhat harder.

From a health standpoint the Missouri River water best meets the health requirements. It normally has excellent clarity and shows adequate chlorine residual in its service areas. Ten Mile water, on the other hand, shows some turbidity at times, and the Board of Health has had a number of tests indicating no chlorine residual in some areas believed to be serviced by Ten Mile water. They have stated that if the Ten Mile System is to be used for a continued supply, additional facilities for improved water treatment must be installed.[18] The State Board of Health has also stated that proper steps should be taken to correct the problem of unsatisfactory chlorine residuals in parts of the distribution system.

The Hale System does not meet accepted health standards because the Hale Reservoir is not covered.[19] Lack of a roof allows the growth of algae and lets birds, insects and other sources of contamination enter the reservoir. The Board of Health has strongly recommended the covering of Hale Reservoir.

Algae is either directly or indirectly a source of tastes and odors in all three of Helena's basic water systems. Although Chessman Reservoir is treated with copper sulfate, algae continues to create taste and odor problems. Decaying algae may cause odors. Oils from decaying algae may react with chlorine to form chlorophenal compounds imparting disagreeable tastes to the Ten Mile water.

Algae is also reported to be a problem at times in the Missouri River source. However, the treatment plant is equipped with a feeder for activated carbon which may be used as a means of pretreatment for taste and odor control.

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[18] Per Mr. Art Clarkson - Montana Board of Health.

[19] Art. 7.0.2. - Recommended Standards for Water Works

CHAPTER IV

NEEDED WATER SUPPLY SYSTEM IMPROVEMENTS

Improvements to the present water supply system are needed if the City of Helena is to insure its residents of an adequate supply meeting health standards and sufficient to provide adequate fire protection. These improvements should take into consideration future needs as well as the existing deficiencies.

The following sections review the problems of the various supply systems and recommends improvements that are intended to correct the existing deficiencies and the costs of making these improvements. Other potential sources of supply are also discussed and a plan for improvement of the supply system presented.

TEN MILE SYSTEM TRANSMISSION FACILITY IMPROVEMENTS

The physical condition of two of Ten Mile's transmission facilities which have caused concern have been treated in detail in this study. These include the Red Mountain Canal and the wooden mains to the Woolston Reservoir.

During the survey of Red Mountain Canal all of the timber trestles were noted to be in an advanced state of deterioration. Failure of a section of this trestle could remove this facility from service during the critical few months it is used, creating a very serious loss of supply. The sheet metal flumes, including those supported on grade, show lack of protective galvanized coating and rusting.

It is recommended that all trestled sections of Red Mountain Canal be replaced with new treated timber trestles and flumes. Those flumes on grade should also be reconditioned by cleaning and painting with a zinc paint.

An economic analysis also revealed that the cost of a corrugated metal cover for the flumes would be paid for over a 30 year period by elimination of snow removal costs. Costs were estimated on the basis of the preliminary construction detail, Figure 2.3. The following cost tabulation is a summary of estimated costs associated with the proposed improvements for Red Mountain Canal.

Remove and replace trestles w/new treated timber and provide new flume	4880'@$20.00 =	$97,600
Recondition existing flumes on grade (Clean and galvanize paint)	6900'@ $2.00 =	13,800
Corrugated galvanized cover	11,780' @$2.00 =	23,560
Estimated Construction Cost		$134,960
+15% Engineering & Contingency		20,240
Total Estimated Cost of Improvements		$155,200

The wooden mains from Yaw Yaw to the Woolston Reservoirs broke in June, 1970 causing a major failure in Ten Mile water delivery. These mains were inspected visually and by performing detailed flow measurements to determine the extent of leakage.

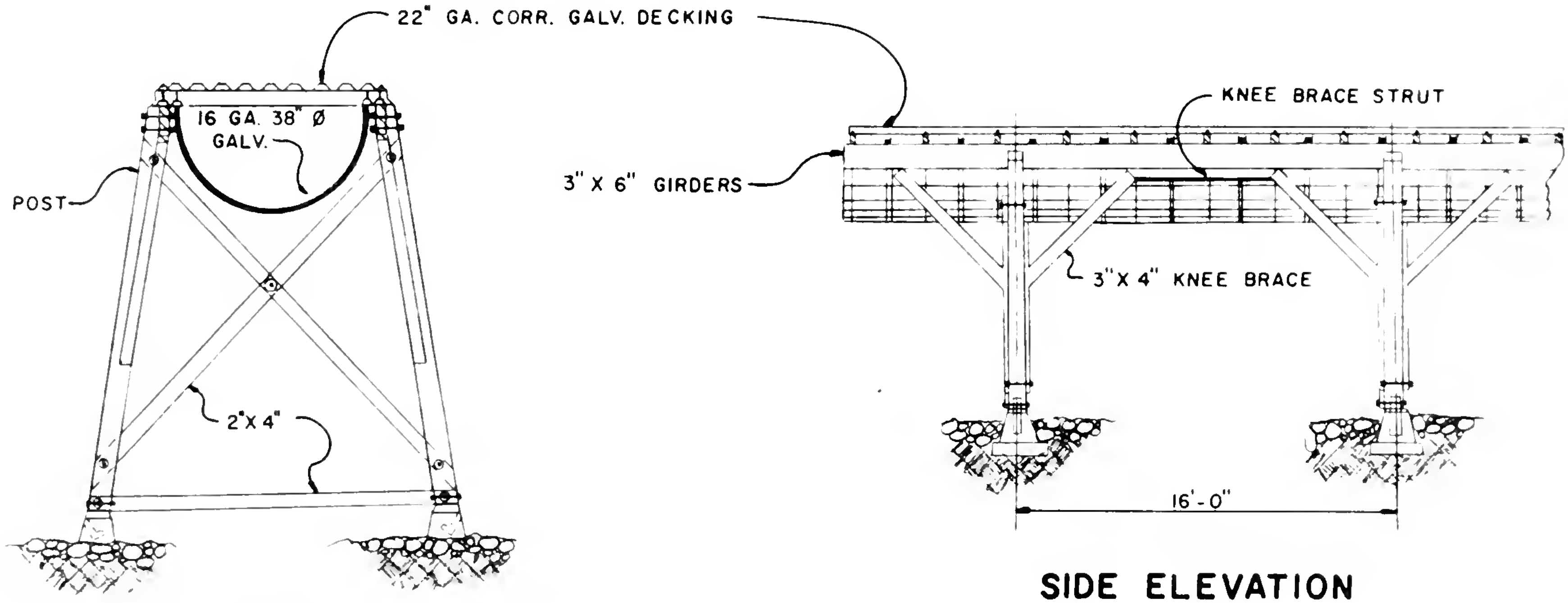
The field investigation conducted July 30, 1970, along the course of the wooden water mains revealed the following regarding the 16" wood stave main:

- Considerable green grass or vegetation growth along the course of the pipe throughout its length.
- Water arising from the ground surface over the location of the pipeline at frequent intervals (100'-300') throughout its length. Evidence of leakage included damp ground, marshy areas, seepage originating over the waterline and ponding to the lower side of the main.
- At two locations seepage originating below the location of the water main collects in the bottoms of drainageways and flows for perhaps one-fourth mile before being dissipated.
- Two locations where the pipeline was exposed and water was spraying from a joint.
- Numerous locations where cover material was eroded away indicating that leakage had occurred and maintenance had been performed.

The field survey of the 24" wood stave main showed:

- Little to no green vegetative growth along the course of the pipeline.

RED MOUNTAIN CANAL



TYPICAL CROSS SECTION

SIDE ELEVATION

TYPICAL FLUME DESIGN PROPOSED
FOR
REPLACEMENT OF EXISTING TRESTLED SECTIONS

Figure 2.3

- No visable wetness was observed originating over the location of this water main.

- Two locations where the pipeline was exposed and maintenance work was visable.

- Two locations where the pipeline failed and was repaired in June, 1970.

While all visable evidence of leakage appeared to originate in the 16" main, it is possible that the 24" main also leaks. In many locations the 16" main is uphill from and adjacent to the 24" main. Where the two wooden mains are widely separated, however, there was no visable evidence indicating possible leakage from the 24" transmission main.

Results of flow measurements taken on the wooden mains were inconclusive in determining leakage losses occurring between the upstream and downstream ends. However, the measurements did indicate that very little water (0.3-0.5 M.G.D.) is being transmitted through the 16" wooden main. This could indicate the presence of excessive silt deposits at low points, obstructions, such as roots, in the pipeline, possible air binding by trapped air pockets at high points, excessive leakage, or a combination of any or all of these factors.

While reviewing methods of measuring leakage losses, detailed flow calculations were conducted on theoretical flows which should be conveyed by all existing pipelines between Ten Mile Settling Basin and Woolston Reservoirs.[20]

These flow calculations were based on the Hazen-Williams equation for discharge (Q)

$$Q = 1.318\ C_h A R^{0.63} S^{0.54}$$

in which C_h is a coefficient of pipe wall roughness, A and R are determined by geometry of pipeline cross sections, and S is determined by pipeline length and difference between static water levels over the length of the pipeline. A high value of C_h indicates a smooth pipeline; capacity is proportional to the value of C_h.

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[20]Re: Estimating leakage losses in water transmission line. Conducted for Northwest Planners by William A. Hunt, PhD. P.E.

In order to equate to nine million gallon per day capacity for Ten Mile System, a value of C_h = 75 was required. The following values are average coefficients of C_h for pipe material:[21]

Cast Iron (40 years old)	-	80
Steel (20 years old)	-	100
Wood Stave (Regardless of age)	-	120

"The flow rates calculated to be compatible with the 6000 gpm total flow figure given by Mr. Drivdahl, the pipe diameters and the difference in reservoir elevations were very low, C = 75. This indicated that the lines should be examined for excessive tuberculation, partial silting, or partially-closed valves at various points throughout the system. The normal value of C for pipes of this type is between 100 and 120."[22]

In view of the investigative work, the following recommendations are made:

- The 16" wood stave main should be taken out of service and abandoned.
- The condition of the 24" wood stave main must be further investigated and found to be in an advanced state of deterioration before its replacement is justified. Other efforts which may be taken to determine if its capacity can be improved follow.
- Investigative work regarding supply capacities must include the steel and cast iron mains between Ten Mile Settling Basin and Yaw Yaw.

The detailed recommendations regarding treatment of the 24" wood stave main only are as follow:

- The pipeline should be uncovered and examined at regular intervals along its entire length. At each location where the pipe is exposed the steel bands should be checked for state of rusting, the exposed wood should be examined for extent of decay, and wood cores should be taken to check the interior

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[21] Handbook of Hydraulics, King and Handbook of Applied Hydraulics, Davis.

[22] Re: Estimating Leakage Losses in Water Transmission Line. For Northwest Planners, August 7, 1970.

pipewall condition and for compressive strength lab testing. If dry-rot of the pipe exterior is extensive, the steel bands will become embedded into the wood allowing the staves to separate and cause the pipe to leak.[23] Only on the basis of results of this investigative work can a decision be made as to whether or not complete replacement of this pipe is justified or whether partial replacement or no replacement is a possibility.

- The ground profile over the pipeline should be plotted and combination air-relief, vacuum-breaker valves should be installed at all high points where no such valves exist. These would eliminate possible air-binding and would protect the pipeline from being drawn inward if vacuum conditions occur. Based on inventory work and hydraulic analysis, it is felt that this pipeline is presently susceptible to vacuum conditions when Woolston Reservoirs are low and water demands are high. Further, based on color slides taken of the pipe exterior during repair, the failure of this pipe in June, 1970, was probably caused by a vacuum condition inside the pipe. Two staves were buckled inward and there was no embedment of the steel bands into the wood.

- Efforts should be made to check for and remove silt deposits, if present, by attempting to open existing blow-off valves and/or by installing new blow-off valves for this purpose. If such silt deposits are present and have become consolidated, tapping the pipe walls and injecting compressed air may break them up so they may be blown off.

To get maximum capacity of flow through the steel and cast iron mains between Ten Mile Settling Basin and Yaw Yaw and to secure needed hydraulic data should design of a new pipeline from Yaw Yaw to Woolston Reservoirs be necessary, the following is recommended.

- Blow-off valves at Yaw Yaw[24] and at the location where the pipelines cross Ten Mile Creek should be serviced or replaced to remove any silt deposits present. Use of compressed air injection may be necessary in accomplishing this. Existing air-relief valves should also be serviced or replaced.

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[23] Mr. Frank Head, Superintendent, Missoula Water System, Montana Power.

[24] Indicated on plan of 24" steel main from City Engineer's office.

- Due to inlet controlled flow, vacuum flow conditions may be present. Their effects should be examined.

- A pressure gage should be installed at Yaw Yaw to determine head loss between Ten Mile Settling Basin and Yaw Yaw at various rates of flow. This would give information on the condition of the pipelines between Ten Mile Settling Basin and Yaw Yaw and the head (energy) available to transmit water between Yaw Yaw and Woolston Reservoirs.

In summary, there is a possibility that the 24" wood stave main can be rennovated to remain in service for ten to twenty years or more and that it could be taken out of service temporarily for repair or partial replacement without danger of collapse. Also, any internal deterioration of the pipe is probably limited to high points along the line where air may have remained in contact with the wood. And, finally, a nine to ten M.G.D. capacity can be maintained after abandoning the 16" wooden main by eliminating flow restrictions as outlined.

If further investigation shows that replacement of the 24" wooden pipeline is justified, the total cost for investigative work, testing, new pipeline construction and servicing supply mains between Ten Mile Settline Basin and Yaw Yaw is estimated to be as follows:

Cost of New 24" Pipeline - 17, 140 L.F.@ $25/L.F.	=	$428,500
Miscellaneous Valves & Fittings (allow 5%)		21,500
Total Estimated Cost for a New 24" Pipeline	=	$450,000
+ 15% Engineering,Administrative, and Miscellaneous		$67,500
+ Servicing Cost (Steel & Cast Iron Mains)		25,000
+ Preliminary Investigation & Testing		10,000
Total Estimated Cost if Replacement of 24" Wood Main is proven to be required.		$552,500

If partial replacement of the 24" wood main is needed, Helena water consumption rates[25] indicate that it could be temporarily

[25] Records of Helena Water Collection & Billing Department.

taken out of service for a six to eight month period from late fall to early spring. This would be possible under the following conditions:

- A cross-town connector were in service.
- The cast iron main from Yaw Yaw to Helena were kept in service.

An expedient construction schedule would allow sufficient time to replace sections as required with the pipe out of service. An alternate method of rejuvenation which may be economically feasible[26] would be the use of a plastic pipe inside the old main. Reaming the wood main or grouting between the wood and plastic would be necessary for a pipe of this size to get the tight fit needed to contain the pressure.

TEN MILE SYSTEM WATER QUALITY IMPROVEMENTS

In treating the problem of inadequate chlorine residuals noted by the State Board of Health, two solutions are possible. The point of chlorination could be placed nearer to Helena in order to decrease the detention time and get disinfecting action deeper into the distribution system. An alternative would be to install better disinfection equipment at Ten Mile Settling Basin and disinfect at higher, better controlled levels. With booster chlorination at two or three key points within the distribution system, this would provide satisfactory chlorine residuals of around 0.1 parts per million at all ends of the system.

With the first alternative, the chlorination station should be located as close to town as possible while providing a minimum of about 30 minutes contact time to allow the chlorine residual to drop to about 0.4 ppm before consumption.[27] Since services are tapped by Dodsonville users on the 14" pipeline just below Yaw Yaw, it would appear that the point of chlorination should be placed about a mile upstream from Yaw Yaw. There are three transmission mains here and chlorinating would require installing three meters and three chlorination injectors. Alternatively, the piping could be changed to bring the three mains into one large pipe

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[26] No cost information was obtained.

[27] Per Mr. Art Clarkson, Montana State Board of Health.

for metering and chlorination and then connected back into the three transmission mains. In either case such an installation would involve considerable expense, including the cost of probable land acquisition and a large chlorine house. In addition, this location would still be a great distance from many parts of the distribution system and would not solve problems of chlorine loss caused by long detention periods in Woolston Reservoirs.

For these reasons, the alternate method of disinfection at Ten Mile accompanied by booster chlorination at a few critical points within the city is favored. This would provide a more satisfactory solution at a probable lower cost. The lower cost is, in part, a result of continued use of the existing Ten Mile chlorine house.

To meet health requirements, the Ten Mile System needs a facility for water filtration. Figure 2.4 indicates a schematic plan for such a facility including the proposed improvements for disinfection.

The proposed facility includes a raw water screen just upstream from the existing settling basin. This would remove leaves, pine needles, grit, etc. from the 18" raw water pipeline and prevent it from entering the reservoir. The existing six million gallon settling basin would remain in service and allow some sedimentation to occur. Possibly some baffling could be installed to prevent short circuiting.

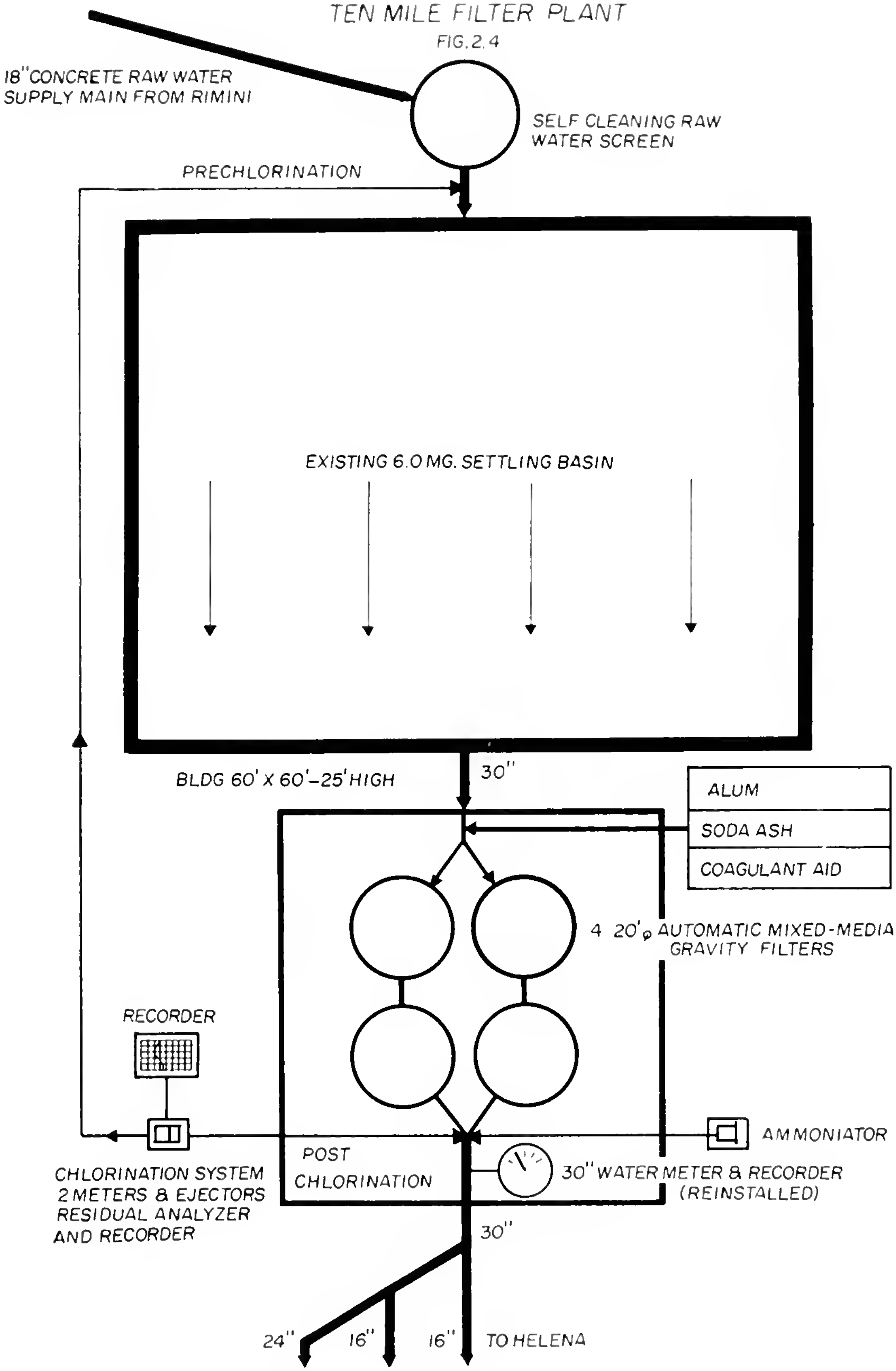
The filter plant would be placed below the settling basin immediately downstream from the present chlorine house. The proposed filters, equipped with dual sand and anthracite media, would provide for high rate filtration of 5.0 gallons per minute per square foot. At this rate it would take about 6,250 square feet of filter area which would be provided by four 20 foot diameter filters. Equipment of this type which has an automatic backwashing feature can be provided, eliminating the need for an operator.

While using filters it may be necessary to provide chemical treatment to coagulate the turbidities for good filtration. Therefore, chemical feeders have been indicated to provide for the addition of alum, soda ash and coagulant aid if needed.

New equipment for improved disinfection and for the removal of offensive tastes and odors would be provided in conjunction

TEN MILE FILTER PLANT

FIG. 2.4



with the installation of this water treatment facility. This would include new chlorination equipment and an ammoniator.

The chlorination system would consist of a new chlorinator for use with one ton containers and a chlorine residual analyzer and recorder. By using two chlorine meters and ejectors, this system would be used to provide prechlorination where the raw water enters the settling basin and postchlorination as the treated water leaves the filtration plant. The level of post-chlorination would be controlled by the chlorine residual analyzer and the recorder would provide a permanent record of residuals in the treated effluent. Postchlorination will prevent the growth of algae in the settling basin, aid in coagulation and may aid in removing tastes and odors at high levels.

The purpose of the ammoniator would be to form chloramines which are relatively stable disinfectants. They react slowly and hence their disinfecting qualities may extend for a considerable distance into the distribution system. Ammonia also serves as a means for taste and odor control.

The existing Ten Mile chlorine house may serve to house the chlorine and ammonia containers and equipment. However, a new building about 60 feet square by 25 feet high would be needed to accomodate the filters. A new chain link fence to enclose the settling basin and new facilities is recommended for protection from trespassers.[28]

A cost estimate was prepared as follows, giving costs to provide a filter plant and disinfection improvements as necessary to bring Ten Mile System to acceptable health standards.

Raw water screen installed	$ 30,000
Automatic gravity filters	175,000
Chemical Feeders	9,500
Chlorination equipment	7,500
Ammoniator	1,500
Building 60' x 60' x 25' high	30,000
Piping and fittings	24,500
Chain link fence	6,000
Subtotal - Construction & Equipment Cost	$284,000
+ 15% Engineering & Contingencies	42,600
Total Estimated Cost for Ten Mile Water Filtration Facility	$326,600

[28] Re. Art. 7.0.3, Recommended Standards for Water Works.

In summary, present needs for Ten Mile System include improvements for Red Mountain Canal and the transmission mains between Ten Mile Settling Basin and Woolston Reservoirs and the installation of a water filtration plant at Ten Mile Reservoir. The following is a summary of estimated costs for these improvements:

Improvements for Red Mountain Canal	$155,200
Replacement of 24" Wood Stave Transmission Main (if found to be necessary)	552,500
Ten Mile Filtration Plant	326,600
Total Estimated Cost All Improvements to Meet Present Needs - Ten Mile System	$1,034,300

HALE SYSTEM

The major deficiency in the Hale System is the need for a roof over Hale Reservoir. Hale Reservoir was constructed by excavating into the natural rock and by extending a stone masonry wall six to eight feet above the excavation. It measures about 132 feet by 142 feet, which would probably need a column and beam system down the center to support a roof system. Since the top of the masonry wall is quite uneven, some type of perimeter beam would be required as well.

A cost estimate was made on the basis of these considerations, which assumed a metal roof deck, supported on steel joists. The estimated cost for installation of a roof of this type, including the columns and perimeter beam, is $3.00 per square foot. The total cost to cover the reservoir is estimated at $56,200.

Some of the small wooden trestles supporting the gravity pipeline from Oro Fino Gulch are in poor condition and may require some restoration, but costs for this were considered as probably minor and were not included for purposes of this reporting.

There is no apparent shortage of supply capacity for the Hale System in meeting peak demands due to the availability of the Eureka well water for such periods.

MISSOURI RIVER SYSTEM IMPROVEMENTS

The Missouri River System is relatively new and generally provides a good quality treated water for municipal use. However,

the Helena water treatment plant does not perform satisfactorily at rated plant capacity. The treatment plant is rated to operate at a 6.0 million gallon per day rate. The Montana State Department of Health has never approved its use at this rate of production. The deficiency appears to be in the clarification (chemical coagulation and sedimentation of suspended solids) of the raw water.

The water treatment plant was evaluated on the basis of physical evidence and on the basis of standards set forth in Recommended Standards for Water Works. These standards which were prepared by a committee of the Great Lakes-Upper Missouri River Board of Sanitary Engineers, are used by the Montana State Department of Health.

Several factors affect the chemical coagulation, flocculation, of suspended material in raw water. Basically, alum is used to react with the alkalinity in water to form an aluminum hydroxide floc. Lime or soda ash may be used to provide the necessary alkalinity and sometime, bentonite is used to increase turbidity and aid in coagulation. In order to properly achieve this, the chemicals must first be thoroughly and vigorously mixed with the raw water (rapid mix) and then more slowly but evenly mixed for a sufficient time to allow the growth of floc which can be readily settled. Proper mixing and adequate detention time for flocculation are very important. Article 4.1.3b. of Recommended Standards for Water Works states "Detention-Minimum flow-through velocity shall be not less than 0.5 nor greater than 1.5 feet per second with a detention time for floc formation of at least 30 minutes."

When operating the Helena plant at a rate of 6.0 M.G.D., calculations indicate a detention time of 11.39 min. in first stage flocculation and 8.62 min. in second stage flocculation for a combined detention time of only 20 minutes. In addition, the dissipation of compressed air for flocculation mixing appears to be uneven and too vigorous to allow the growth of large, easily settled floc. Use of air for mixing could also be causing the collection of minute trapped air bubbles on floc thus giving buoyancy to the floc and making it impossible to settle out.

Study of the sedimentation basins revealed that these facilities also are of insufficient size to allow proper time for sedimentation of floc.

Evidence of poor clarification is clearly visible with a high carry-over of floc into the collection troughs normally seen. This high floc carry-over is also evident when watching the filters during backwashing.

In regard to filtration, the filters should accomodate a plant output rate of 6.0 M.G.D. This is based on criteria of 2.0 gpm (gallons per minute) per square foot of filter area which is a standard maximum for a conventional rapid sand filter. The eight filters provide 2,160 square feet of filter area, which at 2 gpm per square foot would equal 6.22 M.G.D. The only deficiency in the filters appears to be the absence of surface washers which are required by Art. 4.2.1.8 of Recommended Standards for Water Works.

Associated with water filtration plants is the disposition of waste water from filter backwashing and sludge from the sedimentation basins. The present method of disposing of such wastes directly into Prickly Pear Creek is deficient.

An item of equipment which is not adequate for all safety purposes required is the surge suppressor connected to the high lift pumping discharge manifold. This device is not designed to control pressure surges induced by pump start-up; and, since it is solenoid operated, it would do nothing to control the water backsurge resulting in the event of a power failure.[29] At no discharge or pump start-up the high service pumps are capable of developing 500+ psi pressure.[30] Normal operating pressure is 250-280 psi.

The pneumatic butterfly valve controlling the plant raw water influent has been reported as a source of trouble. This valve is held closed by air pressure and breakage of an air line allowed this valve to open when the plant was shut down. Sounding of an alarm allowed the plant operator to take steps to prevent raw water from overflowing the basins and spilling into the pump gallery.

Another area of concern is the raw water piping between the raw water influent wier and the rapid mix basins. Apparent excessive head loss in this piping has caused spillage of water into a waste sump at rated plant capacity. In order to prevent this, sideboards have been installed on the overflow wier.

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[29] Contract documents (Schedule 2-Water Treatment Plant, Sec. IV-J.)

[30] Re: Rating curves for high-lift pumps.

Calculations indicate that partial submergence of the raw water influent wier is probable at a flow rate of about 7.0 M.G.D. indicating the possibility of a marginal condition at 6.0 M.G.D., the rated plant capacity.

Since a wier is calibrated to measure flow while spilling freely, partial submergence would undoubtedly create error in metering the raw water into the plant. It should be noted that records of this metered flow are the basis of payment for water purchased from the U.S. Bureau of Reclamation.

Improvements are needed to eliminate treatment plant deficiencies and bring the Missouri River System up to acceptable standards. Figure 2.5 shows the proposed improvements for the Helena water treatment plant. The improvements include addition of new equipment, structural modifications and piping modifications.

In treating the problem of deficiencies in clarification, it is proposed to remove the concrete partition wall between the rapid-mix basins and the first stage flocculation basins and to install mechanical flocculation agitators. All of the enlarged basins may then be used for flocculation, thus increasing the detention time. The mechanical mixers should then provide satisfactory mixing and avoid problems of air flotation of the floc.

An alternate method for improving flocculation which should be considered prior to final design of plant improvements is the use of other chemicals to improve coagulation. Sodium aluminate fed in conjunction with aluminum sulfate (alum) may be used to aid in formation of aluminum hydroxide floc. If this were used, recarbonation (addition of carbon dioxide) would be advisable to restore insoluble carbonates to bicarbonates and prevent the deposit of carbonates on the filter sand or in the pipe distribution system. A number of commercially made coagulation aids (flocculation aids) are also available which improve coagulation by the formation of a more satisfactory and rapidly settling floc.

Prior to final design a series of jar tests is recommended to evaluate methods of improving flocculation. Air mixing should be further studied by examining floc particles under a microscope to determine presence of minute air bubbles. Also study should be given to determine if air can be dissipated in such a way as to give a more uniform, less aggressive mixing action.

Pilot testing of methods for improving coagulation should be conducted prior to final design and selection of any new equipment.

It is proposed to perform rapid mixing in the deep stilling basin where the raw water enters the plant. (See Fig. 2.5) To achieve this, a modification of the inlet piping would be required to bring the water to the surface of the basin for mixing. A new concrete underflow baffle would be required to still the water for passing the influent wier and to prevent short circuiting.

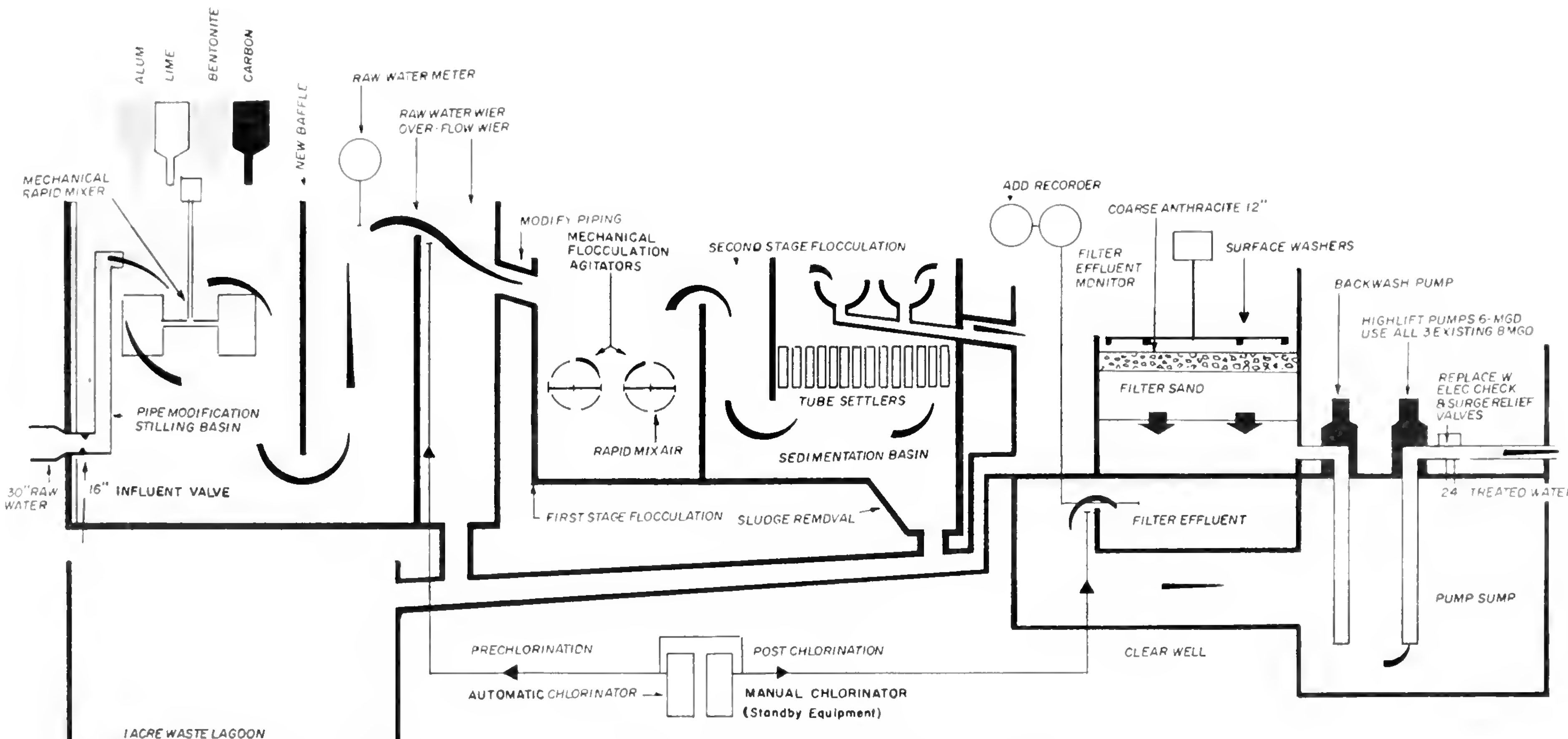
A satisfactory solution for improving sedimentation without physical expansion of the basins would be to install tube settlers in the existing sedimentation basins. These devices have come into use recently and have proven to be an economic means of upgrading a plant to high rate production. Their egg-crate construction provides multiple small openings which slope upward and expedite the collection of sediment on the sloped surfaces. Pilot testing may be necessary before installing tube settlers, but generally surface rates of about 3 gpm per square foot can be expected based on surface-area of the basins. For the 2,116 square feet of surface area of the existing basins, this would achieve a maximum capacity for the basins of about 9.5 M.G.D.

Replacement of the piping between the wier overflow box and flocculation basins with a largersize with better elbows is proposed in order to reduce head loss and prevent submersion of the raw water wier for flows which may exceed 6.0 M.G.D.

Improvement of surge control for high-lift pumping is also recommended. This could best be done by replacement of existing check valves located on each pump discharge with an electric check valve and surge relief valve. At pump start up, electric check valves open very slowly allowing the water to begin its movement gradually. When pumps shut down, or in the event of a sudden power failure, the electric checks close very slowly. The surge relief valves provide a by-pass and relieve water from backsurge overpressures at pump shutdown. The existing surge suppressor might be usable in combination with the proposed surge control improvements.

To eliminate the possibility of the plant influent butterfly valve opening in the event of compressed air failure, it should be replaced with a valve which would automatically close in the event of a power failure.

Another item of equipment which has been troublesome is the chemical elevator, and a better one should be installed to replace it.



PROPOSED IMPROVEMENTS WATER TREATMENT PLANT FIG 2.5

A lagoon consisting of two one-half acre cells is proposed to accomodate waste and allow solids to settle out in lieu of wasting directly into the creek.

Since no record is presently being kept of the volumes of treated water actually pumped to high services from the plant, the installation of a continuous graph recorder with daily totalizer should be considered. This would record discharge over the filter effluent wier. It would provide a better record of water delivered to Helena and also a record of losses by backwashing and cleaning.

The following cost schedule gives estimated costs for proposed improvements needed to correct Missouri System deficiencies and provide satisfactory operation at a 6.0 M.G.D. capacity.

New Mechanical Equipment Installed	$ 47,250
Tube Settlers Installed	45,000
Surge Control Improvements	11,250
Raw Water Control Improvement	4,000
Pipe Modifications	5,000
Structural Modifications	3,500
Waste Disposal Lagoon	9,500
Subtotal - Cost Improvements	$125,500
+15% Engineering & Contingencies	18,825
Total Estimated Costs Improvements for Missouri River System	$144,325

While these improvements would not increase the Missouri System capacity beyond 6.0 M.G.D., the pre-treatment improvements, plant piping, and supply piping would accomodate somewhat higher capacities. The filters would then be the only facilities which would not provide increased capacity. However, by removing approximately twelve inches of filter sand and replacing it with coarser anthracite, higher filtration rates could be achieved. Pilot testing should be conducted to verify this. By doing this and by operating all three existing high lift pumps simultaneously, the plant would provide treated water to Helena at a rate of about 8.0 M.G.D. The pumps would not be operating at their peak efficiencies and no pump would be available for standby purposes if this were done. While this may be satisfactory to meet peak demands, it would not be a recommended practice for steady use. It would be highly advisable to provide the filter modification with the other plant improvements to furnish a higher rate which would be available to meet peak demands.

The cost for remodeling the filters, as stated, is estimated to be $6,000. This is very minimal in rendering possible an 8.0 M.G.D. capacity. The total cost of the Missouri River System improvements would then be as follows:

Estimated Cost of Basic Improvements	$125,500
Cost to Modify Filters to Dual Media	6,000
Subtotal Cost - Recommended Improvements	$131,500
+15% Engineering & Contingencies	19,725
Total Cost to Provide 8.0 M.G.D. (limited use)	$151,225

FUTURE WATER SUPPLY SYSTEMS NEEDS AND SOURCES

Planning to meet future water needs for the City of Helena must be aimed at obtaining water in sufficient quality and quantity from proposed sources as well as insuring that treatment, transmission and distribution facilities will be adequate to supply water at peak rates required by forecasted future demands. In general, the distribution system must be sized to handle momentary peak demands. By providing adequate reserve storage within the distribution system, these momentary peak demands may be met without a transmission facility sized to handle these short duration high use periods. If treatment and transmission facilities are sized to provide sufficient water required for a peak day, depletion of reserve storage should not occur.

Based on present per capita consumption rates and projected populations to be serviced, including those areas of projected growth outside the present city limits, required water transmission capacities of 20.0 M.G.D. and 22.0 M.G.D. have been estimated for 1980 and 1990 respectively. Since the combined capacities of existing systems is about 16 M.G.D., plans should be made to expand capacities by 4 M.G.D. in the next ten years with an anticipated further expansion of 2 M.G.D. for the next ten year period.

Projected annual supplies which must be available are estimated at 3,400 million gallons for 1980 and 3,700 million gallons for 1990. This will mean an increase of 800 M.G.D. per annum for 1980 over the present 2,600 M.G.D. An additional 300 M.G.D. per annum for 1990 will be required.

In selecting future sources economics is the basic criteria governing choice of sites. All costs involved in bringing the water to acceptable quality standards should be included in this evaluation.

Ten Mile System
Water rights restriction and economics appear to severely limit the possibilities of increasing the supply of the Ten Mile System even though there are prospects of capturing additional water from spring snow melt to increase total annual supply from this source.

The City of Helena has by court decree, in Case No. 4989, acquired first rights to 550 miners inches (8.90 M.G.D.) of water from Ten Mile Creek. The following is a quotation from Case No. 4989: "Helena Water Works Co. has the right to divert of the waters of Ten Mile Creek only sufficient water to supply its present pipeline, at its present capacity."

All reports indicate that the waters of Ten Mile Creek are excessively over-appropriated. It would seem then that to increase the supply from this source a new pipeline extending from mountain storage all the way to Helena would be required. Such a facility would be very expensive and would achieve little increase in available annual supply. Therefore, it is recommended that future improvements on Ten Mile System be limited to maintaining the existing system's capacity.

Travis Basin
In order to increase annual supply, two sites for increased storage were studied. The first site studied, Travis Basin, is situated adjacent to the Continental Divide at the headwaters of Ten Mile Creek. (See Figure 2.6) The area is quite remote and lies mainly on U.S. Forest Service land. A field survey of the area indicated a site favorable for dam construction where water could be collected in a gently sloping basin.

A Forest Service map showed a parcel of privately owned land in the creek bottom. In researching ownership deeds, it was discovered that while this had been a placer mining claim originally belonging to Sam Travis, the City of Helena acquired a sufficient portion of the land through condemnation proceedings in August to December, 1953, for reservoir purposes. The following quotation was taken from court records:

> "This Court duly and regularly found, after hearing, that the use for which the property of the defendent is sought to be appropriated, to-wit: Reservoir purposes for the storage of water for the inhabitants of the City of Helena, Montana, for domestic and irrigation, is a public use, within the meaning of the laws of the State of Montana."

A later quotation states "said plaintiff may take, use, and appropriate the property hereinafter described for the uses and purposes for which said land has been condemned." The record indicates that $35,250 was paid to the defendents for the purchase of this land.

As an aid in evaluating the site, Fairchild contour maps (1947-1948) were obtained from the Montana Highway Commission through the City-County Planner. These were used in determining drainage area and estimating storage volumes for a potential reservoir. Run-off data for Ten Mile Creek at Rimini as well as snowfall records for the area from the offices of the U.S. Geological Survey were also obtained. Since no stream gaging has been done at the Travis site, yield was estimated by correlating April snowpacks and Rimini run-off records over a 32 year period to obtain estimated average yield per square mile for the months of high run-off, April through June. Assuming that decreed water rights of downstream users did not preclude collection of run-off for the months of high flow, April through June were assumed, an average annual storage potential of about 240 million gallons from the measured watershed area of 1.95 square miles was calculated.

Investigation of dam requirements for Travis site indicated that a dam of approximately 50 feet maximum height by 750 feet in length would contain approximately 350 million gallons, equal to the estimated yeild for a year of heavy snowfall. A cost estimate was prepared assuming an earthfill dam with a spillway, bottom drain and other necessary hardware. The construction cost for such a dam, including timber clearing for the reservoir, was estimated at $225,000. Adding 15 percent for engineering and contingencies, the total estimated cost for a reservoir facility at Travis site becomes $258,750.

In the economic evaluation of future water sources, an attempt has been made to prorate costs to unit costs as was done in evaluation of present sources. For evaluating future sources an interest rate of six percent applied for 20 years has been assumed as added to each initial cost in comprising fixed costs. Assumptions for life of any potential future facility are the same as were used for present facilities.

In evaluating a reservoir facility at Travis Basin, only capitalized costs for the reservoir plus a small allowance for operating and treatment costs were included, since flow will be entirely by gravity and since other Ten Mile facilities would be available in any case to accommodate this source.

Unit fixed and variable costs have been estimated by dividing estimated annual fixed and variable costs by the estimated average annual storage volume of 240 million gallons. The site appears to have excellent economic feasibility based on the following estimated unit costs:

Unit Fixed Cost = \$9730 ÷ 240 =	\$40.54/M.G.
Unit Variable Cost = \$1500 ÷ 240 =	6.25/M.G.
Total Estimated Unit Cost of Water	\$46.79/M.G.

It is recommended that stream gaging be conducted at the Travis site before any positive plan to develop this water source is adopted to insure storage potential. A stream gage should be set up in the immediate future in order to collect yield data for the Travis Basin Watershed in the spring of 1971.

Scott Reservoir

The other site studied for its potential to increase the future supply of Ten Mile System is Scott Reservoir. Increased storage capacity by building up the existing dam and raising the spillway was studied.

The feasibility of such a project can only be justified if the watershed is capable of significantly greater yield than the capacity of the existing reservoir. The watershed yield potential, based on the same criteria used for the Travis site, was estimated. By these methods it was estimated that run-off collection for the months of April through June would provide an annual average of 230 M.G. available for storage. Since this is only slightly greater than the present 194 M.G. reservoir capacity, the feasibility of obtaining increased supply from this source would seem poor unless further study revealed better watershed capability.

In the course of field surveys, it was observed that on June 12, 1970, Scott Reservoir was full and water was passing the spillway at an estimated rate of 6-8 M.G.D. Flow over the spillway continued until about mid-July (½" depth over spillway on July 15). This was about the same time as demands by downstream water users required taking Red Mountain Canal out of use. However, 1970 snowpack was reported to be above average.

Although the feasibility of increasing the dam height appears to be poor, an estimate of increased reservoir capacity was made assuming a ten foot increase in dam height. By doing this the capacity would be increased by 90 million gallons for an estimated cost of \$115,000.

Gaging of spring run-off over the spillway of Scott Reservoir could be used to give better data for watershed yield capacity.

In addition to studying individual future Ten Mile supply sites, it is pertinent to make a general economic evaluation of Ten Mile system as a future supply source. Its continued use as recommended will involve costs in restoration and a new treatment facility. Costs for the treatment facility must be included in estimating future units costs for water from Ten Mile, while restoration costs in part reflect only the reinstatement of a facility whose life has expired. Included in future initial costs was that for the construction of a new reservoir at the Travis site. Based on a new average estimated annual supply of 1,800 M.G., the following future unit costs are computed from the prorated annual costs. It is assumed that operating costs will remain about the same while chemical costs will increase.

Unit Fixed Cost = \$104,860 ÷ 1800 =	\$58.25/M.G.
Unit Variable Cost = \$44,038 ÷ 1800 =	24.47/M.G.
Estimated Total Future Unit Cost	\$82.72/M.G.

This indicates a projected increase of \$9.92 from the estimated present unit cost of \$72.80 per million gallons for the study period to 1990.

Missouri River System

While the Missouri River System is presently an essential facility for supplementing annual supply, its high variable cost indicates that its use is most appropriate in meeting peak demands.

Available supply is presently adequate for the Missouri River System. Water rights are for 5,680 acre-feet or 1,850 M.G. of water, and the highest annual use to date has been 625 M.G. Using 1,850 M.G. per year combined with an estimated future available supply of 1,800 M.G. from Ten Mile System, assuming development of the Travis Reservoir site, a total annual volume of about 3,650 million gallons should be available from the two sources. This would nearly meet the projected need of 3,700 M.G. for 1990. To do this the supply rate of the Missouri System would have to be increased to 10 M.G.D. by 1980 and to 12 M.G.D. by 1990 to meet estimated peak demand rates.

While a detailed plan has been studied whereby a capacity of 8.0 M.G.D. could be achieved, essentially by correcting existing deficiencies in the Helena water treatment plant, no mention has been made of requirements for providing higher rates. In order to increase capacity beyond 8.0 M.G.D. the following would be required:

- Increase of the treatment plant capacity.

- Addition of a new transmission main from the treatment plant to Helena.

The first item could be accomplished either by physical plant expansion or by limited plant expansion and high-rate filtration. Recent articles on the subject show high-rate methods in water treatment to be most economical. Therefore, a cost estimated has been based on this technique. To use this method it would be necessary to add flocculation and rapid-mix basins, modify plant piping, change to mixed media filter material, modify control and electrical systems and add mechanical equipment including pumps.

The existing 24" transmission main to Helena would not accommodate flows at excess of 8.0 M.G.D. This is because increased friction losses would mean pumping at higher pressure, making power costs excessive and endangering the structural safety of the pipe.

Costs for plant improvements and a new pipeline to connect into the Helena distribution system near the intersections of Montana and Helena Avenues were estimated as follows for a capacity of ten to twelve M.G.D.

Construction Cost Plant Improvements*	$485,000
Construction Cost New Pipeline to Helena	270,000
Subtotal Estimated Construction Cost	$755,000
+ 15% Engineering & Contingencies	113,250
Total Estimated Cost to Increase Missouri River System to 10-12 M.G.C. Capacity	$868,250

*Including $125,500 to correct existing plant deficiencies.

In prorating these costs to unit costs, it is estimated that variable costs would remain about the same as at present plant capacity. Fixed unit costs prorated at a practical plant capacity of 10 M.G.D. are calculated from a new estimated annual fixed cost as follows:

Estimated Unit Fixed Cost = $89,535 ÷ 3,650 =	$24.53
Estimated Unit Variable Cost (same as present) =	111.24
Estimated Total Unit Cost Water = Missouri River System Increased to 10-12 M.G.D.	$135.77

This represents a decrease of $4.15 per million gallons from the present estimated unit cost of $139.92 per million gallons for water from the Missouri River System.

OTHER POTENTIAL WATER SOURCES

In addition to studying methods of increasing the capacity of the Missouri River System and improving the annual supply of water available from the Ten Mile System, several areas were studied for potential development of new supply systems.

Trout Creek

Groundwater emerges in the form of numerous large springs from the granular creek bottom in the vicinity of Vigilante Campgrounds located about six miles east of the town of York or about 25 miles from Helena. These springs form the headwaters of Trout Creek during the dry months of the year when the creek bed above the springs is dry.

During the course of this study, a field trip was made to this site to examine its potential for development as a future municipal supply for the City of Helena. Rough measurements were taken on July 21, 1970, to estimate rate of flow of water discharged by the springs. A flow-rate of about 575 miners inches or 9 M.G.D, was estimated at the time. Water quality appeared to be excellent.

As a first step in determining the feasibility of this water source for future supply purposes, Trout Creek water rights were researched. The stream was found to be an adjudicated stream (York Mines Corp., Pltf. - vs - Louis Rotwitt, Def. - Case No. 16631, May, 1942). There are a total of eight adjudicated water rights decrees. The first two are for all the water in Trout Creek while five others add to 775 miners inches of water. Not all of these have been put to beneficial use. Nevertheless, the following applies to obtaining a water right:

> "To obtain a water right on an adjudicated stream, one must petition the District Court having jurisdiction over the stream for permission to make an appropriation. If the other appropriators do not object, the Court gives its consent and issues a supplementary decree granting the right subject to the rights of the prior appropriators."[31]

A preliminary cost estimate was prepared for a gravity supply pipeline to Helena. A 30" pipeline was estimated to be capable of carrying about 9 M.G.D. of water by gravity to Helena. The approximate cost for such a facility was estimated to be about

[31] Surface Water Rights, State Water Resources Board.

$6,000,000. An alternate 24" pipeline accompanied by pumping (400 HP) revealed a slightly lower estimated initial cost of $4,600,000 but would involve power and pump maintenance costs.

Based on the apparent high initial cost and present interest rates for recovery of capital, in addition to the question of obtaining water rights, this source is not recommended by this study as a feasible 1980 or 1990 water supply source for Helena.

Maupin Creek Reservoir Site

A brief study was conducted for a potential reservoir site on Maupin Creek, a tributary of McClellan Creek. This site, in the vicinity of Strawberry Butte is located about 12 miles to the southeast of Helena. Its development would involve a ditch of about three-fourths mile in length to divert water from one small watershed into an adjacent basin giving a combined drainage area of about 2.8 square miles. A dam and gravity pipeline would be necessary for developing the site for the City of Helena. The cost for placing a 6.0 M.G.D. capacity pipeline, dam and canal and acquiring property was estimated to be about $1,250,000. Using an estimated average annual supply of about 250 million gallons, the capitalized initial cost, 6% for 20 years, prorates to a unit cost of about $182 per million gallons of water. Allowing for variable costs a unit cost of about $200 per million gallons could be expected.

On the basis of this evaluation, this site does not appear to be economically feasible for development by the City of Helena, but may be usable by East Helena to supplement their McClellan Creek facility.

Ground Water Potential

At the request of Northwest Planners, the City-County Planning Board obtained a reconnaissance investigation through the Montana Bureau of Mines and Geology entitled, "Availability of Ground Water for Municipal Use in Helena, Montana" prepared by M. K. Botz, hydrogeologist.

This study provides information regarding development of ground water resources as an alternative to developing additional surface water sources for future municipal use in Helena. The report concludes that a number of areas near Helena have good potential for development of high-capacity wells. These areas are outlined on a map with a legend of potential well yields in gallons per minute.

To determine the feasibility of developing a ground water supply for Helena, an economic comparison should be made between use of wells and pumps and developing surface water sources. In order to do this, two well sites have been selected for study. Cost estimates have been prepared for each. Fixed costs include test wells, production wells, pump facilities and pipelines to connect properly to the water distribution system. Variable costs include pumping costs, operating and maintenance costs and chlorine costs for disinfection, if necessary. Costs for water softening were excluded assuming that ground water of only moderate hardness would be encountered.

Site #1

The first site studied is the vicinity of the State Fairgrounds. This site is favored for the following reasons:

1. It is near the confluence of Ten Mile and Seven Mile Creeks, which may be conducive to good yields by properly developed wells.
2. Its location is good for minimal hazard of ground water pollution.
3. Its proximity to Helena minimizes piping costs.
4. An existing 12" pipeline to the abandoned Woolston well provides a nearby point of connection to the water distribution system.

Based on the potential yeilds, indicated on Plate 1 of the "Helena Ground Water Report", 2 wells in the vicinity of Site #1 as indicated on Figure 2.6 were assumed to give yields of 500 gpm each. Three wells were used to provide about 2.0 M.G.D. which were considered to be about the practical limit of the 12" connection main.

The following costs were estimated for the development of a 2.0 M.G.D. well supply from Site #1:[32]

Test drilling & Pumping	$ 6,500
Three wells & pumping plants	61,000
Chlorination equipment	6,000
Piping cost to system	60,500
Subtotal cost	$134,000
+ 15% Engineering & Contingencies	20,100
Land, Easements, Legal Fees	6,900
Total Estimated Cost 2.0 M.G.D. Well Facility	$161,000

[32]M.K. Botz, "Availability of Ground Water for Municipal Use in Helena, Montana".

Converting this initial cost to an annual cost, capitalized at six percent for 20 years, and assuming a 25 year life for pumps and a 50 year life for wells and piping, gives an estimated annual cost of $6,738 per year. The following unit costs are estimated with fixed costs prorated pumping steadily at a practical capacity of about 1,000 gpm = 1.44 M.G.D.

Variable Costs	
Power Cost	$35.00/M.G.
Operating Cost	10.00/M.G.
Chlorination	2.00/M.G.
Estimated Unit Variable Cost	$47.00/M.G.
Prorated Fixed Unit Cost = $\frac{\$6738}{525}$	12.83/M.G.
Estimated Total Unit Cost Water Used Steadily at Practical Capacity	$59.83/M.G.

These estimated unit costs indicate that if wells at this site can achieve assumed yields, which is dependent on results of test drilling and pumping, developing this site for a ground water supply appears to be very economically favorable.

Site #2

The other site studied is in the vicinity of the present Helena water treatment plant as indicated on Figure 2.6. This site was considered for development of a 4 to 6 M.G.D. well supply. It was chosen for study for the following reasons:

1. Proximity to an existing section of 20" pipeline adjacent to the airport allows connection to the Helena distribution system without undue piping lengths.
2. Potential yields in excess of 750 gpm are indicated on Plate 1 of "Helena Valley Ground Water Report".[33]
3. The area is in a location where ground water level should remain quite stable.
4. It is located away from most sources of potential ground water pollution.

Based on a potential well yield of 1,000-1,500 gpm, a cost estimate was assembled based on three wells and pumping plants to provide a 4 - 6 M.G.D. capacity. The following costs were estimated for such a facility:

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[33] M.K. Botz, "Availability of Ground Water for Municipal Use in Helena, Montana".

Test drilling and pumping	$ 6,500
Three wells, pumping plants, controls	109,000
Chlorination equipment	10,000
Pipeline to Helena	270,000
Subtotal Cost	$395,500
+ 15% Engineering & Contingencies	59,325
Land & Legal Fees	20,175
Total Estimated Initial Cost	$475,000

Unit water costs for Site #2 were computed as for Site #1 with a practical assumed pumping rate of 4 M.G.D., which for steady use would provide 1,450 M.G. of water per year. The following unit costs were estimated for this criteria of use.

Variable Costs	
Power cost	$ 40.00/M.G.
Operating Cost	10.00/M.G.
Chlorination	2.00/M.G.
Estimated Variable Unit Cost	$ 52.00/M.G.
Prorated Fixed Unit Cost = $\frac{\$19,878}{1,460}$	13.62/M.G.
Total Estimated Unit Water Cost Operated at Practical Capacity (4 M.G.D.)	$ 65.62/M.G.

Based on unit costs, this site also appears to be an economic water source contingent upon results of test pumping.

FUTURE SUPPLY SOURCES - RECOMMENDATIONS AND PLAN

In planning to meet the future annual water needs and future peak supply rates for 1980, a Recommended Plan has been prepared as shown on "Future Supply Sources" map (Figure 2.6. Two alternate plans have also been prepared in addition to the Recommended Plan and are shown as Alternate Plan #1 and Alternate Plan #2 on the "Future Supply Sources" map.

The following tables indicate the supply systems that comprise each plan. Capacities of each system, costs of improvements or new construction for each, and estimated unit costs at capacity are listed.

Costs - Water Sources Plans for 1980

Recommended Water Sources Plan

Planned Source	Planned Capacity M.G.D.	Estimated Costs Total Cost	Unit Cost/M.G.
Ten Mile System - Correct present deficiencies & develop Travis Reservoir	9.0	$1,293,050*	82.72
Missouri System - Correct present deficiencies & increase capacity	8.0	151,225	135.12
Hale System - Correct present deficiencies	1.0	56,200	
Develop well Site #1	2.0	161,000	59.83
Totals	20.0	$1,661,475	

Alternate Plan #1

Planned Source	Planned Capacity M.G.D.	Total Cost	Unit Cost/M.G.
Ten Mile System - Correct present deficiencies & develop Travis Reservoir	9.0	$1,293,050*	82.72
Missouri System - Correct present deficiencies	6.0	144,325	143.75
Hale System - Correct present deficiencies	1.0	56,200	
Develop well Site #2	4.0	475,000	65.62
Totals	20.0	$1,968,575	

Alternate Plan #2

Planned Source	Planned Capacity M.G.D.	Total Cost	Unit Cost/M.G.
Ten Mile System - Correct present deficiencies & develop Travis Reservoir	9.0	$1,293,050*	82.72
Missouri System - Correct present deficiencies & increase capacity	10.0	868,250	135.77
Hale System - Correct present deficiencies	1.0	56,200	
Totals	20.0	$2,212,500	

Note: Costs as herein tabulated are estimated from current prices. Allowances for changes in cost index must be made at a future date if substantially different.

*Includes cost of $517,500 for complete replacement of the 24" wood stave transmission main if proven necessary by further investigation.

The Recommended Plan was selected because cost estimates showed it to have the lowest estimated initial cost as well as generally lower estimated unit water costs. The Missouri System at an increased capacity of 8.0 M.G.D. was included because study showed this capacity to be very economically achievable after correcting existing deficiencies. It was also felt to be an essential facility to supply Helena when a year of relative drought is encountered. At the same time, its apparent high variable costs for supplying water would indicate that its use should be limited to supplying peak demands as much as possible.

Alternate Plan #1 has a somewhat higher initial estimated cost. It could be used in lieu of the Recommended Plan if well Site #1 does not prove capable of providing expected yield. Although estimated initial costs of Alternate Plan #1 are higher than for the Recommended Plan, unit costs indicate that over a period of years it would provide water to Helena at approximately equal costs as the Recommended Plan.

Alternate Plan #2 is offered only as a solution if ground water by well supply cannot be obtained at a reasonable initial cost. Increased initial costs for a well supply would at a point increase prorated unit costs beyond those calculated for water supplied by the Missouri System at an increased capacity of 10 M.G.D. This point was calculated to be at an initial cost for a well supply which would exceed $2,271,000 assuming recovery of capital at six percent for 40 years. The break even point would be higher for a shorter recovery period, but ability to raise increased annual payments in order to do this may be impossible for a project of this large an initial cost.

The rennovation and improvement of Ten Mile System, at its present rate of supply, was included with all plans considered. Estimated costs of necessary improvements and new construction appear to be quite reasonable for a system of 9.0 M.G.D. capacity as do estimated future unit costs for the water. A reservoir facility at the Travis site appearsto be justified by the estimated relative low unit cost of $46.79 for water supplied. This was therefore included in planning for future annual consumption needs.

Water supply for 1990 could be achieved by combined capacities of proposed well sites or by other appropriate combinations of the sources indicated on the "Future Supply Sources" map (Figure 2.6.

If the Recommended Plan for 1980 were implemented and well Site #1 proves satisfactory, then planning for 1990 needs by increased well supply at Site #2 is recommended because of its economic feasibility. Well supplies yielding a combined capacity of 4 M.G.D. together with an increased Missouri System capacity of 8.0 M.G.D. and the existing capacities of 9.0 M.G.D. and 1.0 M.G.D. for Ten Mile and Hale Systems, respectively, would give a total supply rate of 22.0 M.G.D. This is calculated to be equal to the maximum daily demand for 1990.

Increasing the Missouri System capacity beyond 8.0 M.G.D. is not recommended except as a last resort.

It is strongly recommended that well supplies be proven reliable by drilling test wells and pumping tests in the immediate future. These test wells should be drilled at suggested Sites #1 and #2. Any other sites selected should be selected recognizing that the primary costs of well supply are piping and pumping costs.

CHAPTER V

HELENA WATER DISTRIBUTION SYSTEM

The primary deficiency of the existing city water system is its inability to properly distribute water. This has caused severe water shortages in two parts of the city. Water rationing on Helena's West Side has been necessary at times because supply shortages threaten to deplete the reserve storage in Woolston Reservoirs. This situation is primarily due to inefficient distribution of the available supply rather than a shortage of supply or inadequate reserve storage.

Records of the Helena water treatment plant operation indicate that on days of peak demand, the treatment plant has not been operated in excess of 16 to 18 hours. This has been necessary because Malben Reservoir has become filled while the water level in Woolston Reservoirs has remained at a critically low level. To correct this, a crosstown connector is proposed to help make the two points of reserve storage act in unison. This would give capability of pumping continuously from the treatment plant at high demand periods and would supply approximately 2 M.G. more to the system (pumping 24 hours vs 16 hours) which would be available on the West Side.

The other area in Helena which has suffered low pressures and water shortage has been the residential area located on Winne Avenue and higher and east of Montana Avenue. This area is presently zoned off by check valves and a small booster pumping installation provides some temporary relief to the area during high demand.

A situation which has not created a water shortage but has been a trouble source is the extremely high pressures encountered on Helena's low north side. Pressures in this area run continually in excess of 140 psi. This high pressure has been a source of frequent pipeline breakage and has therefore involved very high maintenance costs.

A final deficiency, already stated in a preceding section of this report, is the inadequacy in chlorine residual at various extremities of the distribution system.

A computerized analysis of the flow conditions in the Helena water distribution system was authorized and conducted in order to determine the improvements necessary to correct deficiencies

in the existing system and to assure sufficient capacity to supply the City of Helena and its environs with water through the year 1990. Another purpose was to determine the rate at which the pumping plant on the Missouri River can supply water to the Woolston Reservoir during periods of minimum flow, as well as to see if the proposed improvements will permit supplying the average maximum daily demand for 1990 from the existing reservoirs when the pumps are not operating. A final purpose was to establish an order of priorities for implementing the improvements to the system determined by this study.

The scope of the study was limited to that portion of the Helena distribution system supplied by Ten Mile and the Missouri River Systems including the Woolston and Malben Reservoirs. The Hale distribution system and that part of the existing syste m lying south of Winne and east of Montana Avenue were not included in the computer analysis, nor did it include recommendations for the physical layout of additions to the system to accommodate the projected population growth in the areas where such growth is anticipated. However, the study does include provisions for supplying water in the quantities projected at points in the existing system adjacent to growth areas. The results of this study are included as Appendix B.

RESULTS OF COMPUTER ANALYSIS

As a result of this computer analysis of the Helena water system, certain improvements are recommended. The more important of these improvements are classified as primary improvements, those that are desirable but not urgent are classified as secondary improvements. The primary improvements, which should be implemented as soon as possible to increase the efficiency of the existing system, include the following in order of recommended priority:

- An 18" diameter line to be installed on Dakota Avenue, a 16" diameter line to be installed on Broadway, a 20" diameter line to be installed to connect the 16" diameter line with the 24" line between Holter Street and Woolston Reservoir. This 20" diameter line should have no intermediate connections with the distribution system.

- A 10" diameter line to be installed on Benton Avenue extending north from Hauser Street, west along Meadow Street to the intersection of Meadow with Brady Drive.[34]

- An altitude control valve to be installed in the line leading from the east side reservoir at Alta and Seminole.

The recommended secondary improvements to be taken under advisement include:

- Increase of the capacity of the line connecting the terminus of the 20" diameter line in the vicinity of the airport to the intersection of Lamborn and Walnut and the increased line eventually be extended to connect across to Montana Avenue.

- Installation of a 10" diameter line on Montana Avenue from Boulder to the Burlington-Northern railroad tracks.

- Installation of a 12" diameter line from the intersection of Roberts and Walnut south under the Burlington-Northern railroad tracks to connect with the 12" diameter line terminating at the depot.

These secondary improvements are not critical in increasing the efficiency of the system as far as supplying the basic needs of the city are concerned. However, they should be considered as a back-up system to provide a supply from the pumping plant in case of failure in the 20" diameter line on Prospect leading from the pumping plant. The existing line between the pumping plant and the north side of Helena is not adequate to provide more than 500 to 600 gallons per minute for the system.

HELENA WATER DISTRIBUTION SYSTEM PLAN

A water system plan has been prepared and is shown in Figure 2.8. This plan is aimed at correcting existing deficiencies and is consistent with the recommendations from the results of the computer analysis of the water distribution system. The details of the recommended improvements are discussed in the following sections.

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[34] The exact location of this pipeline in the Sunhaven vicinity can be adjusted to fit the redesigned street locations currently being planned without changing its effectiveness to the system.

Cross Town Connector

Computer analysis indicated that a cross town connector on Broadway would provide potential for pumping into Woolston Reservoirs at a rate of 1,994 gpm during periods of low demand (2.2 M.G.D.) using only two of the existing three high lift pumps at the Helena water treatment plant. It also showed very marked improvements in the distribution system under a variety of demand and use conditions including improvement for fire protection. The water system plan, therefore, has included the cross town connector on Broadway as a means of correcting the existing west side shortage and providing for 1990 needs.

In order to properly operate the cross town connector, a system including a control valve and electrical equipment will be required. Figure 2.9 is a schematic showing the probable control features which would be required to achieve satisfactory operation. Controls include an altitude valve at Malben Reservoir and a water level transmitter at Woolston Reservoirs. The transmitter-receiver would be necessary to monitor the water level in Woolston Reservoir at the Helena water treatment plant so the operator could stop pumping when reservoirs became filled.

To accompany this a continuous graph recorder is recommended to plot the water level in Woolston Reservoirs. This would be valuable in providing information on demand rates for peak and off-peak periods of operation. A telemetering system for Woolston Reservoir level to be monitored at Ten Mile Settling Basin is also recommended to assist in regulating flow rates from that supply source.

Also included in the program for installation of the cross town connector was a chlorination station for booster chlorination of water leaving Woolston Reservoirs. Some pipe adjustments near Woolston Reservoirs are necessary.

The following cost estimate was prepared for providing piping, control system, pipe connections and adjustments, and a booster chlorination station.

Costs - Cross Town Connector

	With Pavement Replacement	Without Pavement Replacement*
11,600 L.F. Pipeline (specified on plan) (including connections, fittings, etc.)	$186,340	$186,340
Booster Chlorination Station & Equipment	10,000	10,000
Regulating System (valves, adjustments & reservoir level indicators & recorder)	31,200	31,200
4,100 LF. Pavement Replacement (Broadway)	16,400	
Subtotal Cost	$243,940	$227,540
+ 15% Engineering & Contingency	36,590	34,130
Total Estimated Cost	$280,530	$261,670

*Applies if reconstruction of Broadway in planning stages by Montana Highway Commission occurs simultaneously with pipeline installation.

Winne High Service System

A preliminary design was prepared for a new high service pumping system to meet present needs in the Winne Avenue area and to provide means for future extensions to projected growth areas as well as to meet projected 1990 demands. The system includes a pumping station (two 300 GPM pumps - 25 HP), a small loop system for efficient distribution and a 300,000 gallon storage reservoir.

The planned system includes providing booster chlorination at the pumping plant. The planned service zone and necessary improvements are shown on Figure 2.8. The following cost is estimated for the improvement:

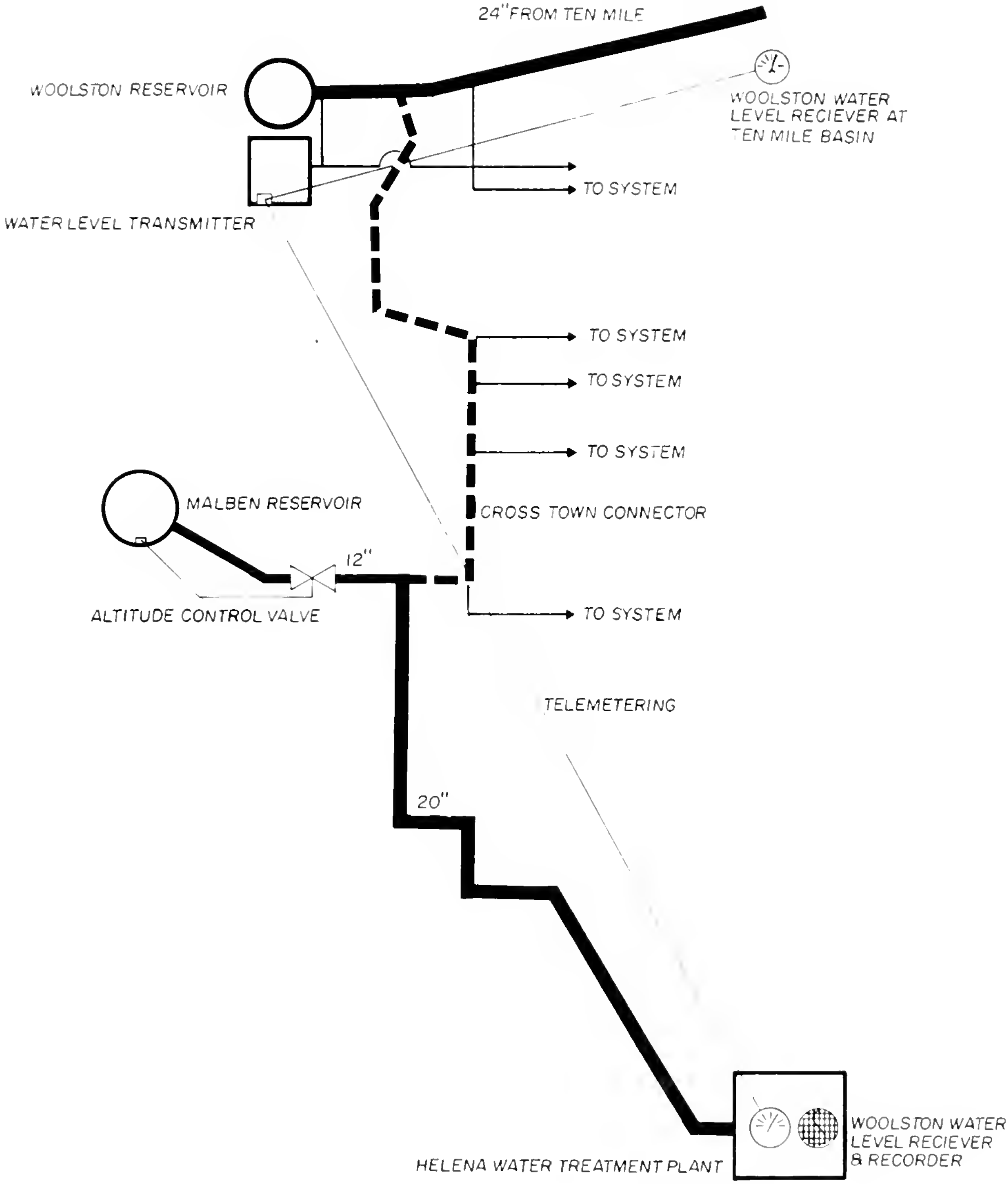
Pumping plant including chlorination equipment	$ 27,000
New pipelines	79,000
300,000 gallon reservoir	21,000
Subtotal Cost	$127,000
+15% Engineering & Contingencies	19,050
Total Estimated Cost - Winne High Service System	$146,050

Planned Pressure Zones

In addition to the proposed Winne High Service Zone, two additional

REGULATING SYSTEM FOR CROSS TOWN CONNECTOR-SCHEMATIC

FIG.2 9



new pressure regulated service zones are proposed to decrease pipe maintenance costs in the Sunhaven and North Side areas as indicated on Figure 8.

The following costs are estimated for installing pressure regulating valves for the planned zones of service:

Pressure Regulation - Sunhaven Zone	$ 5,000
Pressure Regulation - North Side Zone	5,000
Subtotal - Pressure Regulation	$10,000
+ 15% Engineering & Contingencies	1,500
Total Estimated Cost - Pressure Regulation	$11,500

Booster Chlorination Stations

In addition to providing booster chlorination at Woolston Reservoir and for the proposed Winne High Service Zone, a booster chlorination facility is recommended for another area in Helena.

To correct occurrences of poor chlorine residual in the vicinity of the County Hospital, north of Sunhaven, it may be advisable to furnish booster chlorination on the 6" supply main near the Municipal Golf Course. This facility is indicated on the Water System Plan. (Figure 2.8)

Cost for this chlorination facility is estimated at about $7,500.

The costs of correcting all of the existing deficiencies is summarized below:

Estimated Cost - Cross Town Connector (inc. items as indicated on schedule)	$280,530
Estimated Cost - Winne High Service System (inc. chlorination facilities)	146,050
Total Estimated Cost - Pressure Regulation	11,500
Booster Chlorination Station	7,500
Total Cost - Correcting Existing Water System Deficiencies	$445,580
Say	$446,000

FUTURE REQUIREMENTS - WATER SYSTEM

To improve service for future projected development in the Sunhaven area and ensure fire protection in the vicinity of Carroll College, a 10" pipe replacement is included among the Primary Improvements in a previous section. Figure 2.8 shows this improvement as part of the future Water System Plan. Costs for a 10" main as indicated are estimated as follows:

7,300 L.F. of 10" pipe in place	$ 58,400
7,300 L.F. of paving replacement	29,200
Estimated Subtotal	$ 87,600
+ 15% Engineering & Contingencies	13,140
Estimated Total Cost Improvements on West Side	$100,740
Say	$100,000

Expansion of the Winne High Service Zone System would appear to be necessary to provide water to the projected areas of development. Assumed improvements for this area are shown by dashed lines on Figure 2.8 . Improvement costs were estimated as follows:

Add 300,000 gallon reservoir	$ 21,000
Replace motors and pumps (with two 500 gpm units)	10,000
7,200 L.F. water main extension	49,000
Subtotal	$ 80,000
+ 15% Engineering & Contingencies	12,000
Estimated Cost - Future Expansion for Winne High Service Zone	$ 92,000

In coordinating the recommended Future Water Supply Plan to the requirements of the distribution system, the improvement of a lower supply leg from the terminus of the existing 20" main by the airport seems appropriate to accommodate the increased supply rate of 8.0 M.G.D. from the Helena treatment plant. This improvement is recommended among the Secondary Improvements. It is shown in dashed lines on the Water System Plan (Figure 2.8).

The following costs are estimated for the recommended north side improvements:

Estimated total cost pipes installed		$ 75,000
Interstate highway crossing - assume		30,000
Railroad track crossing - assume		25,000
Estimated Subtotal Cost		$130,040
+ 15% Engineering & Contingency		19,506
Estimated Total Cost North Side		$149,546
Water Syste m Improvements	Say	$150,000

The total costs of improvements needed to meet future needs are summarized below:

West Side (Benton Avenue) improvements	$100,000
Expansion - Winne High Service Zone	92,000
North Side system improvements	150,000
Estimated Total - Future Improvements	$342,000

The total cost of correcting existing deficiencies and meeting future needs is broken down as follows:

Summary - Total Costs 1980 Water System Plan

Estimated Cost - Correct Existing Deficiencies	$446,000
Estimated Cost - Meet Future Needs	342,000
Estimated Total - 1980 Plan	$788,000

Note: All estimated costs in this report are based on current prices. For future uses, costs must be adjusted if the cost index has changed significantly.

RECOMMENDED PRIORITIES FOR WATER SYSTEM IMPROVEMENTS

The priorities listed below have been established according to urgency of need, state of deterioration of existing facilities and the needs for a logical sequence of construction.

1. Cross town connector on Broadway and Winne High Service System.
2. Restore Red Mountain Canal and Ten Mile transmission mains and construct a filtration plant at Ten Mile Settling Basin.
3. Provide booster chlorination stations within the Helena distribution system. (Winne High Service Zone, Woolston Reservoirs and Helena Northwest Side)

4. Correct Missouri treatment plant deficiencies with increased capacity to 8.0 M.G.D.
5. Provide Sunhaven and Helena Northside pressure regulation zones. (See Figure 2.8)
6. Construct West Side distribution system improvements (10" main on Benton Avenue to connect into system on Brady Street.)
7. Construct reservoir at Travis Basin site.
8. Construct well supply - Site #1 (for 1980 demands)
9. Construct North Side system improvements. (see Figure 2.8)
10. Provide expansions for Winne High Service System.
11. Increase well supply - construct Site #2. (to meet 1990 demands)

The cross town connector and high service system have been given equal priority since both improvements are necessary to correct present residential water shortages. Ten Mile System proposed improvements are also felt to be of approximately equal importance and are therefore placed at the same priority level.

JURISDICTIONAL AREA WATER FACILITIES

Individual wells and pumps supply most of the present water in the jurisdictional area. There are, however, a number of relatively small joint facilities. These have been inventoried and are indicated in map form on Figure 3.2. These joint facilities are generally well and pump installations with elevated storage reservoirs or ground storage reservoirs. Well and pump facilities are used to supply water for the Masonic Home, Mountain View School and Bonanza Packing.

A limited number of joint users outside the Helena City Limits obtain their water through extensions from the Helena water supply system. Included among these are Fort Harrison, the Fairgrounds and the County Hospital north of Helena.

East Helena which lies immediately outside the jurisdictional area to the east has its own supply. Facilities include a 730 gpm gravity system taking water directly from McClellan Creek and a well and pump system supplied by two 600 gpm capacity wells in the valley north of East Helena. (Figure 3.2)

Future Needs in the Helena Valley

One part of the jurisdictional area which may have potential future need for both a water and a sewer system is the Helena Valley. This area extends for several miles to the north of the city limits.

Ground water is encountered at shallow depths throughout the Valley and is a very economical source of water supply to its residents. Generally a 50 to 60 foot depth well and a small pump and pressure tank provide an adequate water supply for a residence.

Because of the present and projected population densities within the area, the question of possible pollution of shallow well supplies has created a need for additional study and planning in order to avoid a possible crisis. There are indications already of potential pollution in the Helena Valley ground water.

In his report, *Availability of Ground Water for Municipal Use in Helena, Montana, A Reconnaissance Investigation*, ~~Dr~~. M. K. Botz states as follows:

> "In my opinion, the most serious potential pollution problems in the Helena Valley are the septic tank-water well systems associated with most of the new housing subdivisions. Septic tank effluent can seriously pollute ground water. There is a thick clay layer between water horizons in some parts of the valley, but the continuity of this relatively impermeable layer is not known. In addition, some shallow wells are taking water from the same strata that are receiving septic wastes. The long-term use of individual septic tanks and water wells will most probably create a pollution problem."

Recommendations of this report constitute a workable plan to meet future needs for water and sewer improvements in the Helena Valley.

Since a joint water supply system for Helena Valley may be necessary if ground water pollution is eminent, a rough design for a water system to serve a projected 1990 population of about 4,000 located in areas of present development has been prepared. (See Figure 2.10)

The system consists of three wells and pumping plants of about 2/3 M.G.D. capacity each, a two million gallon reservoir located in the Scratchgravel Hills to the west of Forest Vale Drive, and a distribution system to service the indicated areas. The wells would have to be of sufficient depth, probably 200 to 300 feet, and properly located to avoid pollution hazards. The system indicated is primarily a schematic model used for demonstration purposes and as a basis for preparing a rough cost estimate. Final design considerations, economics and other factors may change the configuration and costs considerably. The following summary gives estimated figures for the system shown in Figure 2.10.

Estimated cost of reservoir	$ 111,200
Wells and pumping plants	73,800
Distribution system and trunk lines	1,321,800
Estimated subtotal cost	$1,506,800
+ 20% Engineering & Contingencies	301,360
+ Land and Legal Fees	30,000
Total Estimated Cost - Water System for Helena Valley	$1,838,160

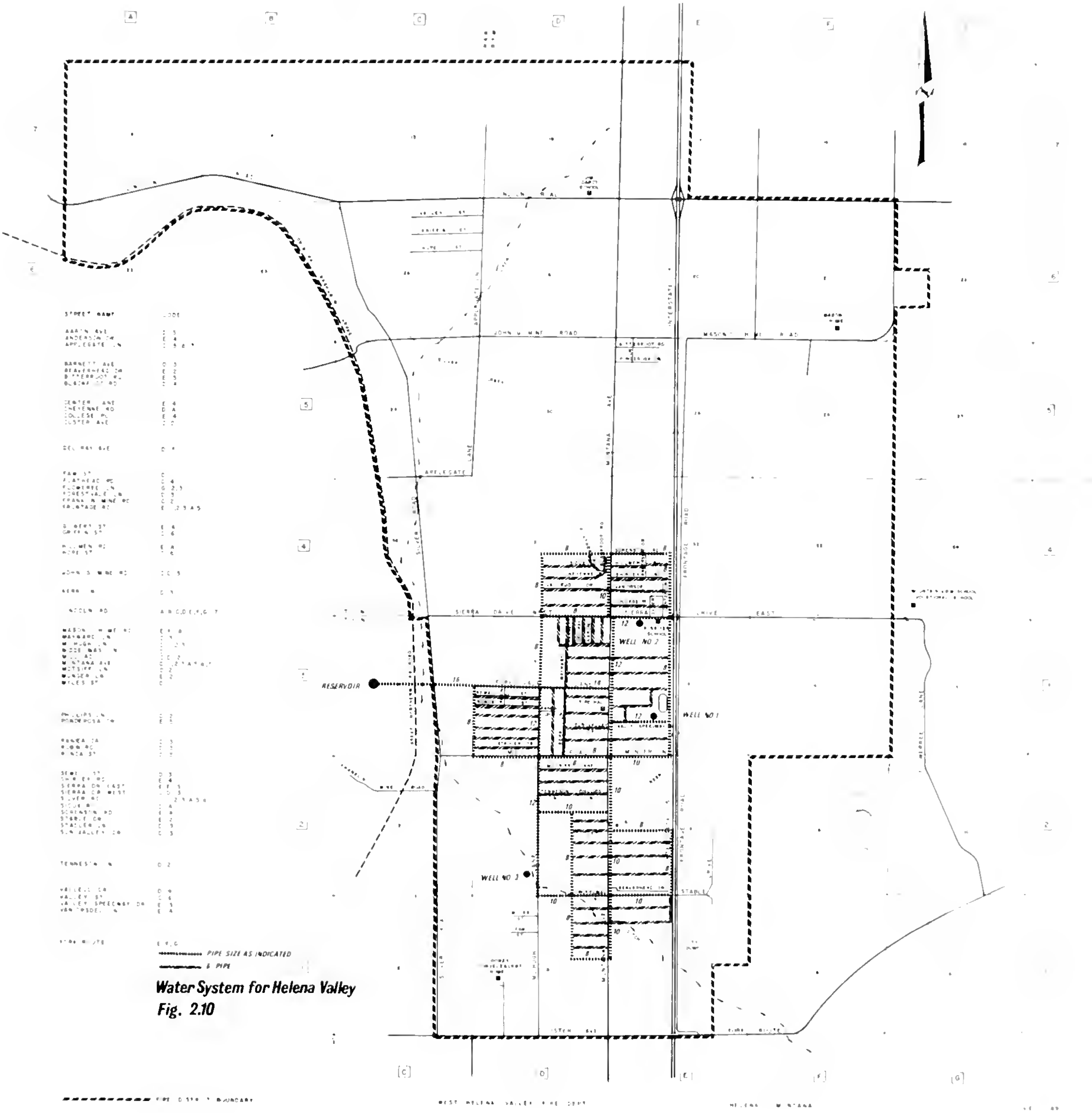
Based on a present population of about 2,000 for the area served, this would amount to an estimated cost of about $920 per person. At this cost, it would appear that at the present density, a joint facility would represent no savings over individual systems. However, if population densities increase and pollution of individual well supplies becomes a major problem, such a facility may be justified.

Future Needs of East Helena

Future water needs of East Helena were studied as they may affect planning within the jurisdictional area. An examination of existing East Helena water supply facilities was conducted to determine present capacities and relate their adequacy in meeting future demands.

In 1960, East Helena considered buying water from the City of Helena. The proposal was to construct a new pipeline from the Helena water treatment plant to East Helena.[35] This plan was abandoned in favor of obtaining water by wells and pumps. The two 600 gpm wells indicated on the Jurisdictional Area Facilities map, Figure 3.2, have since been constructed. These wells perform very satisfactorily with only 15 to 20 feet of drawdown at rated capacities.

[35] Preliminary Studies and Report for Water System Improvements, East Helena, Montana, Morrison-Maierle, Inc., Helena, Montana.



Water System for Helena Valley
Fig. 2.10

By use of these wells and the 8" gravity pipeline, a combined supply rate of 1,930 gpm is provided. With steady pumping (24 hours per day), this would provide 2,780,000 gallons per day or by pumping 16 hours per day would provide 2,203,000 gallons per day. The following table was compiled using present and projected population figures as given in the Comprehensive Area Water and Sewer Plan for Lewis & Clark County by Intermountain Planners, Billings, Montana, and the estimated daily available supply from existing water supply facilities.

East Helena
Available Per Capita Supply - Present Facilities

Study Year	Estimated Population* East Helena	Pumping 24 hr/day G.P.C.D.	Pumping 16 hr/day G.P.C.D.
1970	1660	1674	1327
1980	2200	1263	1001
1990	2750	1010	802

*Includes East Helena fringe area.

Based on a maximum daily demand of slightly over 700 G.P.C.D. computed as the combined land use consumption rate for the City of Helena, it would appear from this table that present East Helena water supply facilities should be more than adequate to meet 1990 water demands. It is, therefore, concluded that the City of Helena should not have to plan for extension of the supply system to provide municipal water to East Helena in the 20 year study period to 1990.

CHAPTER VI

SANITARY SEWER SYSTEM

The purpose of this report on the sanitary sewer system for Helena and its Jurisdictional Area is to examine the present status of the system and make recommendations for an updated comprehensive sewer plan. The chapter discusses the general history of the system, examines its deficiencies and makes recommendations for correction of existing problems and for the long range improvement of the entire city system as well as those systems in the Jurisdictional Area.

In many cities such as Helena, early sanitary sewer systems evolved as the city grew. The first sanitary disposal system consisted of outdoor privies and cesspools. As these became overloaded and outmoded, the easiest course of action was to connect to a convenient storm drain. The first sanitary sewer system evolved into a combined storm drain and sanitary sewer, growing as the city grew. This is evidenced in Helena by the records indicating storm inlet and drain connections to the sanitary sewer system in the vicinity of the Central Business District, the oldest part of the city.

The records showing the location of sanitary sewers are kept and maintained in the City Engineering Department located in the Civic Center. Plan location records of sewers are generally complete, except for some older sewers, the profile elevations for storm drain connections, older manholes and grades. Depths of manholes and the distances between them are on record, but elevations for tops, inverts and grades are not recorded for many old lines. Sewers constructed under Special Improvement Districts or extended in the past 20 years have been properly recorded as to elevation and plan location. Plan locations of all known sewers have been transferred from the records to a sewer base map for the entire city by the City Engineering Department. The map reproduced on a reduced scale is included on Figure 3.1 of this report. Analysis of the sewer system showed the sewers are generally in good to excellent condition.

Many of the city's records indicate connections of storm inlets and drains to sanitary sewers. The records are incomplete, making the use of them difficult. The records are sufficient, however, to indicate general areas where combined sanitary and storm sewers exist. This evidence of combined sewers can be seen through storm drain records and the problems commonly associated with combined systems. These problems include:

overloaded sewers backing up into private systems; flooding at the treatment plant; overflowing manholes; bypassing of diluted raw sewage around the treatment plant; deposition of silt and grit; and flushing of surface debris to the treatment plant. Cases exist where the connection of storm inlets and drains to sanitary sewers have been eliminated during street construction have taken place but were not recorded.

The city has a policy of separating the two systems when new construction occurs. The Helena Urban Renewal Program will also contribute greatly to the elimination of roof drains and miscellaneous connections from storm inlets into sanitary sewers.

Prior to 1960 there were two outfalls for Helena. The one known as the Cleveland Street Outfall was located on the west side with discharge going to the Ten Mile drainage near the county hospital. The other was located on the east side with discharge going to an open ditch near the present sewage treatment plant. A new 24" concrete outfall sewer from the Cleveland Street line was run parallel to Custer Street and connected to the plant during the latter part of 1960.

A small number of residences within the city limits are presently using septic tanks and drain fields for sewage disposal. These cases exist because of extreme topography which makes it impractical to connect into the sanitary sewer.

Extensions and additions to the present sewer system within the city limits are made by three methods. One is by the city having contractors install short sections to serve property owners upon their request, with the respective property owners paying. The second is installing extensions at the time of construction of street improvements. The last is through the formation of Special Improvement Districts (see Figure 3.4).

ANALYSIS OF EXISTING SYSTEM

The construction of the treatment plant for the City of Helena was completed in 1960. At this time the plant was equipped with a Parshall flume in the inlet structure which measures flow passing through the plant. A continuous recorder is located in the control building. It uses a circular seven day chart and is capable of measuring from 1.00 M.G.D. to 10.0 M.G.D. Also at this time the Cleveland Street and the east side outfalls were combined at the treatment plant for treatment and metering.

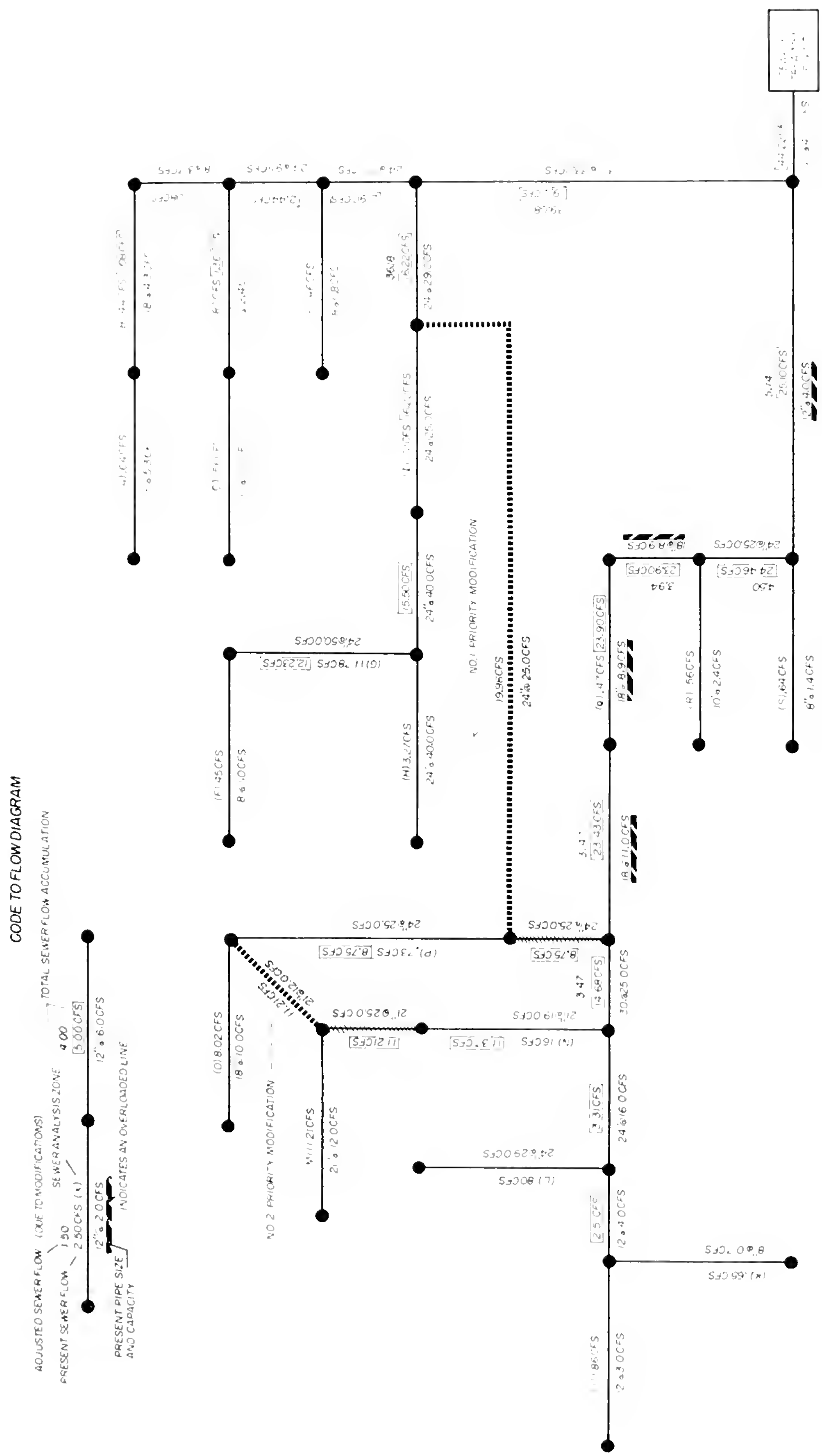
This provided a method for keeping a continuous record of total sewage flow going through the plant. Prior to 1960 no records could be kept. The Parshall flume, however, is not able to measure bypassed sewage.

In order to analyze and evaluate the capacities of the sewers it was necessary to divide the area being served by the Helena sanitary sewer system into convenient tributary areas or analysis zones. Each zone represents an area that covers or serves a system of laterals and branch sewers which collect the sewage from the respective areas and discharge at a common point, taking into account flows from adjacent and/or higher areas.

The tributary areas presently being served were defined as the result of topography, economic factors and need for service. Since very little of the city remains to be served, future tributary areas within the city limits will probably be defined by topography, while those outside the present city limits by topography and political boundaries. Tributary areas used for evaluating the sanitary sewer system are shown on Figure 3.6.

Peak flows of sewage from the tributary areas were computed and totaled as they collected from area to area on the way to the treatment plant. The theoretical capacities of sewers flowing full were compared with the computed flows within. The system was then analyzed for capability of meeting its immediate and future needs within the city limits along with future needs for limited expansion outside the city. For this evaluation it was assumed that areas having a population density of less than five people per acre were impractical to serve. The analysis zone population figures were taken from the report, "City of Helena and Jurisdictional Area Population Projections 1968-1990", prepared by the City-County Planning Department.

To obtain the average daily water demand for tributary areas used in the analysis it was necessary to total the prorated amounts from each segment of each water analysis zone within the tributary area. The water systems average daily flow, calculated for each water analysis zone, is based on population, floor space, land use and impact projects. The results of the analysis of the sewer systems requirements inside the city limits are shown in Table 1. Figure 3.7 is a schematic diagram of the system showing theoretical capacities of sewers flowing full with computed flows expected within. Table 2 shows the results when those areas indicated outside the city limits are included.



SEWER SYSTEM SCHEMATIC FIG. 37

Design criteria used for the analysis:

Average daily flow	=	50% x average daily water demand
Peak daily flow	=	2 x average daily flow (based on flow records)
Infiltration	=	16.7% x average daily flow
Storm runoff contribution: Via manhole covers	=	10% of total storm runoff contribution (1.9 M.G.D.) distributed uniformly over entire system
Via roof drains, inlet connections and miscellaneous	=	90% of storm runoff contribution (17.1 M.G.D.) distributed over areas shown in Figure 3.5
Peak combined flow in 1964	=	24.0 M.G.D. (partially measured by plant recorder)
Estimated peak combined flow in 1990	=	28.5 M.G.D.

All existing sewers are root and sediment free for full free flow.

<u>Flow Records</u>

Flow records start in the latter part of 1960 and are continuous except for breakdowns in the recording equipment during the years of 1965, 1966 and 1967. Charts on file at the treatment plant indicate that in four different years the recorder's capacity was reached or surpassed by the flow through the plant. The duration of the peak flows was less than four hours and most often less than three.

The sewage treatment plant is hydraulically designed to begin bypassing diluted raw sewage at 6.0 million gallons per day. The flow recorder has a capacity of measuring up to 10.0 M.G.D. In four out of the ten years the recorder has been in operation, maximum flows have exceeded the 10 M.G.D. capacity.

The average daily flow for the first year of operation was approximately 1.0 million gallons per day and has gradually increased to 2.8 M.G.D. Because of occasional storm runoff and increase in ground water infiltration, the average daily flow increases during the months of April through September in an amount as much as 100 percent more than the average daily flow during the winter months.

Infiltration changes so gradually that is is necessary to summarize daily flow records for the entire year and compare to see the change. Storm runoff is indicated on the charts as abrupt peaks of short duration, normally less than four hours. These abrupt peaks occur during the months of May, June, July and less frequently in April and September. Severe thawing and snow melt runoff is sometimes indicated on the charts but not to the extent of the spring and summer rain storms.

Excluding the effects of storm runoff, it is found that the average daily peak flow each month is 50% greater than the average daily flow for the same months. When storm runoff is included, the average daily peak flow during applicable months increases to 100% greater than the average daily flow for the same month.

The peak flow reaching the sewage treatment plant was computed by using various hydraulic formulas for the flow from the Parshall flume back through and including the diversion box located at the south property line of the plant. The measured flow passing through the plant recorder was approximately 10.0 M.G.D. and calculated flow going over the bypass weir at the diversion box was 14.0 M.G.D., for a total flow of 24.0 M.G.D. In evaluating the sanitary sewer system 19.0 M.G.D. was attributed to storm runoff, with ten percent or 1.9 M.G.D. of this assumed as entering through manhole covers for the entire system and the remaining 90 percent, 17.1 M.G.D., entering via roof drains and storm inlet connections within the general area shown on Figure 3.5.

<u>Relationship of Sewer Flows, Water System Demands and Population</u>

The average daily sewage flow for 1970 was estimated at approximately 2.8 M.G.D. with a contributing population of 22,730. Of 2.8 M.G.D. flow, 16.7% or 0.467 M.G.D., is assumed to be ground water infiltration. Subtracting infiltration, the average daily contribution would then be 2.333 M.G.D. for the 22,730 people, or 103 gallons per capita per day.

The average daily flow for the time period from 1960 to 1970 increased from 1.9 M.G.D. to 2.8 M.G.D., while the population increased from 20,227 to 22,730. Allowing 16.7% of the average daily flow for infiltration, the average daily contribution per capita increased from 78 gpcd in 1960 to 103 gpcd in 1970. This is an increase of 25 gpcd for the ten year period. Allowing a 25 gpcd increase per decade, it can be expected that the average daily contribution may increase to 153 gpcd in the next 20 years.

The present average daily water demand is 330 gpcd. If 50% of this is used for the average daily sewage contribution, it would be 165 gpcd compared to the estimated 153 gpcd noted above. For this study 165 gpcd was used for the average daily sewage contribution which is 50% of the average daily water demand. The annual average daily minimum flow in 1960 was 1.11 M.G.D. which increased gradually to 1.80 M.G.D. in 1970.

Deficiencies of Existing Sanitary Sewer System

Analysis of the existing sanitary sewer system indicates that, except for one outfall on the east side, the system is capable of meeting its flow requirements until 1990 when the predicted future growth within the city limits and areas adjacent to the east and west boundaries develop.

The one outstanding deficiency determined from the analysis is the outfall line running north from the Burlington-Northern Railroad depot on Harris Street, to Cole Avenue, east along Cole Avenue to Oakes, north on Oakes Street to Cherry, northeasterly across the interstate highway, north along the interstate highway to the York Road (Custer Avenue) and east to the sewage treatment plant inlet line.

This outfall is badly overloaded by as much as six and one-half times during periods of major storm runoff. The 12" portion of this outfall is overloaded by normal maximum sewer flows without storm runoff contribution. See Figure 37 .

To consider and recommend a construction program for the elimination of storm runoff connections to the sanitary sewer system in this report is impractical for the following reasons:

- Tracing out and isolating each storm drain connection would be laborous and difficult. It would require one or more combinations of smoke and/or dye detection methods be applied in a comprehensive field investigation project.
- The results of the field investigation would have to be analyzed and evaluated throughly from an economic viewpoint to justify such a program.
- The frequency of major storm runoff occurrences is not enough to justify the large cost expenditure that would be involved to separate the systems.

EVALUATION OF JURISDICTIONAL AREA SEWER NEEDS

The existing sewerage facilities within the Jurisdictional Area outside the Helena city limits is limited to a small number and size of existing public systems. Records are scattered about in files of private owners, governmental agencies and consulting engineers. Known locations are shown on Figure 3.2. These systems service individual public facilities including those at the county hospital, Fort Harrison and the Mountain View School. There is also a private system to service the Treasure State Acres Subdivision. (See Figure 3.2) Other than these public systems, those remaining are generally private residences with septic tanks and drain fields.

Within the Jurisdictional Area there is a wide variety of topographic features that range from the mountainous terrain along the south and west edge to the relatively flat Helena Valley in the northeast third. Except for the Seven Mile drainage in the northwest corner of the area, the average drainage flows northeasterly and eventually all drainage within the area finds its way to the west end of Lake Helena. See Figure 3.3

The Jurisdictional Area adjacent to the city limits, south of the railroads, is relatively steep and falls in a general northeasterly direction. North of the railroad properties the land flattens considerably, but still falls to the northeast.

The present location of the sewage treatment plant is such that all of the city and this area adjacent to it can be served by gravity sewers. A large portion of the land adjacent to the city on the east and west sides, yet south of the railroads, is also capable of being served by gravity flow. See Figure 3.3 On the other hand, a considerable amount of pumping by sewage lift stations would be required to take sewage from the Helena Valley part of the Jurisdictional Area to the site of the present sewage treatment plant. The ideal location for a sewage treatment facility to serve the remaining portion of the Jurisdictional Area with gravity sewers lies in the northeast corner of the area near the confluence of the Prickly Pear Creek, Ten Mile Creek and the west end of Lake Helena.

Future sewer construction in the Helena Valley may encounter areas of ground water because of the levels varying from eight feet to seventy feet. Sewage lagoons or oxidation ponds constructed for future treatment facilities should be located in low areas to permit gravity flow of sewage. It can be

expected that they will have to be constructed under wet conditions and be sealed with clay lining material because of the gravel formations which are common throughout the area.

The largest potential need for sewage facilities in the area outside of the Helena city limits is that part of the Helena Valley lying north of the city limits and west of the interstate highway leading to Great Falls. This area has and will continue to develop rapidly. The present population is approximately 2,000 people with a projected population in 1990 of 4,000. The average property is residential. Most of the homes have shallow wells for water supply. It is obvious that as the density of population increases, so does the possibility of contamination of the ground water by septic tank drain fields. Once the ground water is polluted, it cannot be predicted how long it will take to return to its normal condition even if the situation causing the pollution is corrected.

A limited number of ground water control tests have been performed by the Ground Water Division of the Water Resources Branch of the United States Geological Survey. Some of these indicate a relatively high concentration of nitrogen compounds (33 ppm). The presence of nitrogen compounds is generally an indication of the presence of organic compounds, which may or may not be attributed to pollution. It is sufficient warning, however, to cause concern. If the nitrogen is in the form of nitrates, there is a potential hazard to the health of infants who might contract methemoglobinemia, which is a disease of infants resulting from the ingestion of nitrates found in water. Water used for formula feeding of babies should not contain more than ten to twenty ppm of nitrate nitrogen.[36]

Sewage facilities in the Jursidictional Area have been limited to the installation of small isolated systems. Generally these systems meet only the immediate demand and are not tied into an overall comprehensive plan that meets future requirements. Unless such a plan is developed and implemented with zoning and other ordinances, this trend will continue. It is difficult to convince people to pay for something they feel they have no need for at present. However, the Jurisdictional Area should be subdivided into smaller sewer analysis districts, since only a complete engineering study and evaluation will determine which and what size districts are practical.

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[36] H.M., et al., Methomoglobinemia and Minnesota Well Supplies, J., American Water Works Association, 42, 161 (1950).

A cursory look indicates that separate districts have merit. Proposed districts should be reviewed from the standpoint of: the areas most easy to promote to make the necessary improvements; the ones that could be tied together physically and most economically for all concerned; how they would affect future growth and development; the topographic and/or physical barriers that would affect the project; what political subdivisions should be considered in delineating the areas; and what is the best route for organizing and financing such projects.

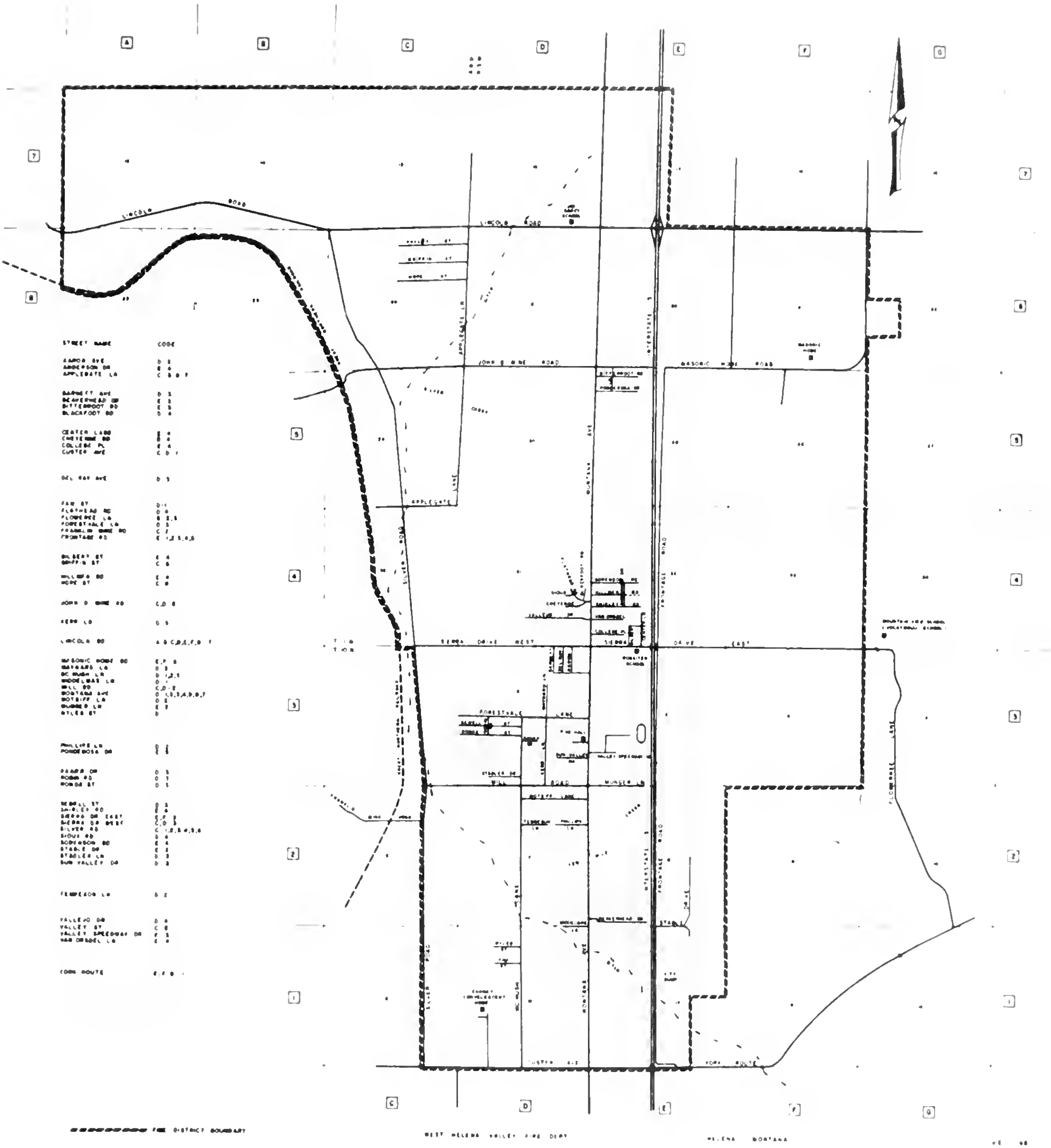
A look at the jurisdictional topographic map points out that gravity sewer systems can be designed and constructed throughout the area. The systems could take the form of major independent sewer districts, complete with collection, outfall and treatment facilities. They also could be planned and constructed in such a manner so that at the end of the design period they would comprise a single district with one treatment plant located near the westerly end of Lake Helena.

The Helena Valley area is the most crucial area of concern outside the city limits. A general schematic sewer system has been prepared (see Figure 3.8) to show how the area could be served by a gravity system. The treatment facilities would be located in the general location of the confluence of Ten Mile Creek and Prickly Pear Creek drainages near the west end of Lake Helena. As shown on Figure 3.3 , it is impossible to use gravity to convey sewage from the Helena Valley area to the present sewage plant treatment. Lifting of the sewage from this area to the existing plant would be required.

To relocate the present sewage treatment plant to the westerly end of Lake Helena is a consideration. This would require an outfall from within the city limits to the proposed new site. This proposal would cost in excess of one-half million dollars. The same problem would be involved in carrying effluent from the existing primary treatment plant to a common treatment facility in the Helena Valley.

RECOMMENDED SANITARY SEWER PLAN FOR THE CITY OF HELENA

The recommended plan for improving the existing sanitary sewer system within the City of Helena includes:



LINCOLN ROAD
APPLEGATE LANE
JOHN G MINE ROAD
MASONIC HOME ROAD
INTERSTATE 15
FRONTAGE ROAD
MONTANA AVE
SILVER ROAD
SIERRA DRIVE WEST
DRIVE EAST
FORESTVALE LANE
MILL ROAD
FLORENCE LANE
STREET NAME
CODE
FIRE DISTRICT BOUNDARY
WEST HELENA VALLEY FIRE DEPT
HELENA MONTANA

- Maintaining present attitudes, programs and ordinances for disconnecting existing storm drain connections to the sanitary sewer system and keep the two systems separated on all new construction.

- Installation of new outfalls to bring sewer service to areas immediately adjacent on the east and west edges of Helena.

- Reducing the overloading of the existing east side outfall from the depot district to the treatment plant described as running north from the Burlington-Northern Railroad on Harris Street to Cole Avenue, east along Code Avenue to Oakes, north on Oakes Street to Cherry, northeasterly across the interstate highway, north along the interstate highway to the York Road (Custer Avenue) and east to the sewage treatment plant inlet line. (See Figure 3.6)

  This recommendation would require implementation of one of the following alternatives:

  - Connect a new 21" line from the existing 21" line running down Helena Avenue to the existing 21" line running north on Montana Avenue. Connect a new 24" line to the existing 24" sewer on Phoenix at Sanders Street and run it north on Sanders to connect to the existing 24" outfall on Cole Avenue. This would reduce the overloading of the outfall line to an amount within its normal expected capacity. This, however, would slightly overload, by 20 percent to 25 percent, the capacity of the 24" sewer east of Sanders on Cole Avenue to the interstate highway crossing and on to the sewage treatment plant. Due to the fact that the sewers are large, they would not present the problems that occur when the existing 12" outfall overloads. The main advantage of this alternate is lower cost.

  - Relieve or replace the overloaded 12" outfall sewer from its connection to the inlet line at the treatment plant south to its crossing of the interstate highway with a larger line of sufficient size to carry the computed flow. Along with the preceding, it is recommended to replace or install a relief line alongside of the 18" outfall from the interstate crossing at Cherry Street west along Cole Avenue to Harris, and then along Harris to Phoenix, to connect to the existing 30" brick sewer coming from across the railroad property. The chief advantage of this alternate is that all outfall lines would be relieved from overloads. Also, the manhole on the east side

of the interstate highway crossing would make an ideal place for the new east side outfall line to connect into from the future tributary area T (highway complex, Liquor Control Board warehouse, etc.).

RECOMMENDED SANITARY SEWER PLAN FOR AREAS ADJACENT TO THE CITY

The west side areas lie adjacent to the west city limits of Helena and are defined as tributary areas U, V and W on the analysis map. (See Figure 3.6)

The elevation and capacity of the 15" sewer on Brady Street is sufficient to serve all three areas by gravity. To serve area "U" it is recommended that a 12" outfall be constructed from the 15" sewer on Joslyn near the south property line of the Burlington-Northern Railroad spur line. The general location is shown on Figure 3.6. Final routing would depend on the availability of easements, right-of-way and best topographic location for economy and ability to serve as large an area possible.

The outfall installed to serve area "W" should be large enough and in such a position that it can be extended to serve area "V". This is necessary because area "V" falls off to the west from area "U".

The east side area to be served is located on the east edge of the Helena city limi s, generally south of the railroad properties and on both sides of the interstate highway. On the analysis area map, Figure 3.6 it is noted as Area "T". It is recommended that an outfall of at least 18" in diameter be constructed to connect into the sewage treatment plant from this area.

The alternative selected to correct the overloaded sewer discussed in the Helena recommendations will largely determine how the recommended 18" east side outfall will be constructed.

- The outfall can originate at the sewage treatment plant and run as shown on Figure 3.6 .
- The east side 18" outfall can be started at the manhole on the east side of the interstate highway at Cherry Avenue and run in the location indicated on Figure 3.6. This will depend upon whether the overloaded

Helena 12" outfall sewer along the east side of the interstate highway from Cherry Avenue to the York Road is replaced with a 24" or greater diameter outfall.

The final location will again depend upon easements, right-of-way availability, topography and ability to serve the maximum amount of area.

The new outfall for this east side annexed area is shown penetrating only a short distance into Area "T" and then running only to the south side of the East Helena Highway. A high percentage of area "T" is undeveloped which makes it a logical place to stop the outfall. The construction of lateral, branch and connecting lines to the proposed outfall would be indefinite as to location and should be part of the financial responsibility of those developing the area and not a general obligation of the entire city. Political subdivisions and topographic features may make it necessary to show the end at some other location.

RECOMMENDED SANITARY SEWER PLAN FOR THE BALANCE OF THE JURISDICTIONAL AREA

The recommended plan for that part of the Jurisdictional Area which does not have an immediate possibility of being annexed into the city limits of Helena or of receiving city sewer and water services is outlined below.

In the areas in the southern part of the Jurisdictional Area which cannot physically or economically be accommodated by Helena's municipal sewer system, it is recommended that independent sewer districts be formed. The sewer systems should be constructed as land development occurs and be geared to the required service. These systems should be comprised of collection, outfall and adequate sewage treatment facilities for complete treatment.

For the part of the Jurisdictional Area known as the Helena Valley it is recommended that the following steps be taken:

- The City-County Planning Board should institute a complete ground water control testing program for the area. Biological and chemical tests should be made to determine if there is contamination. Frequent testing in main locations is necessary to completely cover the area to indicate problem areas.

The study could be performed by consulting engineers or by the Ground Water Division, Water Resources Branch, United States Geological Survey. If the county requests the study be made by the USGS possibilities of financial assistance by the agency may become available. Property owners within the area may have to bear costs not covered by the USGS.

- It is recommended that a County Water and Sewer District be formed in accordance with Chapter 45 of Section 16 of the Revised Codes for the State of Montana, with legal council retained to assist in the formation of such a district. When the sewer district is formed as a legal entity, qualified to transact business through a Board of Directors, a complete and thorough design study of both water and sewer improvements of areas selected by the Board should be made. Such a study is not within the scope of this particular report because it will entail a great deal of field survey, delineation of areas to be served and more detailed cost estimates to base financing and bond elections on. The Board of Directors could select a plan deemed best for implementation, whether it be sewer or water improvements separately, or a combination of the two.

Two additional methods for obtaining sewer and water system improvements for Helena Valley are possible:

- Create and form a "Rural Improvement District" in accordance with Chapter 16 of Section 16 of the Revised Codes for the State of Montana.

- Create and form a "Metropolitan Sanitary Sewer District" in accordance with the provisions of Chapter 44 of Section 16 of the Revised Codes for the State of Montana. This can be accomplished if the City of Helena and Clark County deem it in their best interests.

The Helena Valley area is still in the development stage, converting from agricultural to residential land use, with much of the land still to be subdivided. This situation makes it difficult to know where streets and public thoroughfares will be platted. The sanitary sewer system shown on Figure 3.8 has been prepared to arrive at an approximate cost estimate for a system with the above situation. The system shown on Figure 3.8 may not bear any resemblance to the final layout plan used for construction and installation.

If a continuous sanitary survey of the ground water supply for the Helena Valley is begun as recommended and evidence of contamination appears, steps should be taken immediately to correct it. Action may be to design and construct either a water or sewer system, or a combination of both. This should be done especially in the event that the contamination can be traced to septic tank disposal fields.

The system to be constructed will depend upon a number of factors including the amount and extent of contamination found and economics.

The most serious problem is the pollution of surface and ground waters of the area. Installing a sanitary sewer system with adequate treatment facilities will reduce or eliminate contamination except that contributed by agriculture. This condition has existed in the Helena Valley the better part of a century with little apparent problems to domestic water supplies. However, as the area further develops and increases in population, the amount of agricultural pollution will decrease and amount of sewage contributed by humans will increase. By placing sanitary sewerage facilities into operation first, pollution of ground water used for domestic water supplies will be reduced to a point that the valley resident population can continue to use private domestic wells for some time to come. Until ground water control tests indicate water supplies unsafe, public water systems should be installed as needed.

Two alternatives are possible for the treatment of sewage from the Helena Valley area:

- All sewage from the area could flow by gravity to a site near the western end of Lake Helena, where it could receive complete treatment, and then be discharged to natural drainages. The preferred method of treatment would be an oxidation pond, sewage lagoon, because of low operational costs and relatively maintenance free operation. However, the area mentioned for the proposed site is close to Lake Helena which is the habitat of large numbers of migratory fowl. Such an installation might bring forth an ecological or environmental problem. If the environmental problems were proved to be true, a mechanical type treatment plant may have to be considered. Possibly an aerated lagoon, designed so that the water surface would have an extremely turbulent nature would discourage water fowl and resolve any problems.

- The sewage from the Helena Valley area could be pumped in stages to the location of the existing treatment

plant, there to be combined with the sewage from the City of Helena and treated. It would require three or four lift stations to do the pumping and approximately 20,000 feet of varying sizes of force main.

COST ESTIMATES FOR SANITARY SEWER IMPROVEMENTS

Cost estimates will be given in order in which they were discussed in the preceding sections. They are based on average present day bid prices. An allowance should be made for inflation until the time of construction. Separate estimates are presented for the individual areas discussed in the preceding sections.

City of Helena Sewer System

Alternate I

Construct a 21" connection at Helena Avenue to the Montana Avenue sewer		$ 5,000
Construct a 24" relief sewer from Phoenix to Cole at Sanders		62,000
	Total	$ 67,000

Alternate II

Replace or relieve the 12" outfall from the sewage treatment plant inlet to the interstate highway crossing at Cherry (24" outfall)		$ 80,000
Replace or relieve the 18" outfall sewer from Phoenix and Harris north on Harris to Cole, east on Cole to Oakes, and north on Oakes to the interstate highway crossing (21" outfall)		73,000
	Total	$153,000

Areas Adjacent to the City of Helena

West Side

Install a new 12" outfall sewer for that area on the west edge of Helena and defined as Tributary Area U on Figure 3.6	$ 53,000
Install new 15" and 12" outfall sewers to furnish service to Tributary Areas V and W.	$160,000

East Side

Alternate I: from the sewage treatment plant inlet line, install a new 18" outfall sewer to the south side of the East Helena Highway as shown on Figure 3.6	$224,400
Alternate II: from the most easterly manhole on the east side of the interstate highway, install an 18" outfall to the south side of the East Helena Highway as shown on Figure 3.6	$170,000

Other Parts of the Jurisdictional Area

The following estimates are based on the Helena Valley sewer system being constructed as one district and treated at one treatment plant. Two alternatives were used in estimating costs. One is to consider developing a Helena Valley Sewer District complete with its own treatment plant and independent from the City of Helena facilities. The other is to form an independent Helena Valley Sewer District with sewage pumping stations to lift the sewage to the existing sewage treatment plant. A revision of this plant would be necessary to accommodate the additional flow.

Costs for the alternatives would be:

Collection system and outfall lines (Annual operating and maintenance cost = $20,000)	$1,400,000
Treatment:	
Independent of the City of Helena's facilities -	
Alternate I (standard sewage lagoon)	$450,000
Annual operating cost = $5,000	
Alternate I (mechanical treatment plant)	$620,000
Annual operating cost = $35,000	
Alternate II (pumping to existing sewage treatment plant and expanding existing, plus proposed secondary future)	
a. Pumping stations & force mains	$290,000
Annual operating cost = $25,000	
b. Modifications to existing primary sewage treatment plant	$250,000
c. Increase in capacity of future secondary treatment addition over that needed for the city's need.	$180,000
Increase in annual operating cost = $20,000	

CHAPTER VII

SEWAGE TREATMENT FACILITIES

The existing facilities for sewage treatment for the City of Helena are classified as primary treatment. Essentially, this is providing a wide spot in the outfall line to slow the velocity and allow solids to settle out. A detailed and comprehensive description of the treatment process follows. Figure 3.9 shows this graphically.

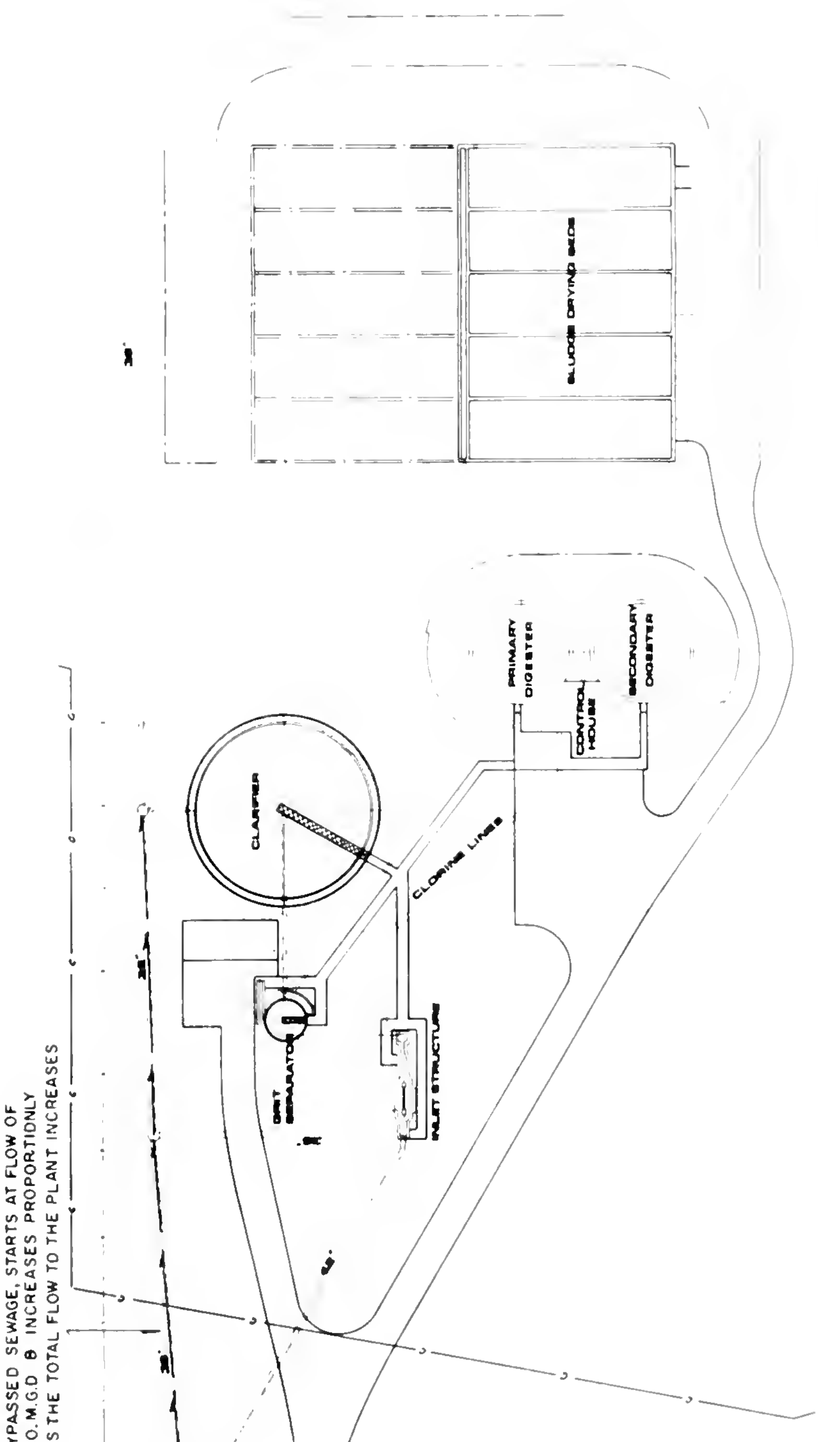
Description of Treatment Process

Sewage enters the treatment plant site at the diversion box located at the south property line. The purpose and function of the diversion box is to automatically bypass diluted raw sewage around the plant at peak flow conditions. This occurs when storm runoff causes the total flow to exceed 6.0 M.G.D. The plant hydraulics are designed in such a manner that at a 6.0 M.G.D. rate of flow, the water surface elevation reaches a weir crest elevation in the diversion box. If flow increases beyond 6.0 M.G.D., part of it flows over the weir.

The sewage that passes through the diversion box, excluding bypassed sewage, flows through a 42" inlet line to the inlet structure. This is a device which provides coarse screening, grinding of solids and metering of sewage.

Sewage then leaves the inlet structure and flows to a grit separator, where the velocity is decreased enough so that sand, gravel and other heavy materials can settle out. Enough velocity remains, however, to transport organic solids through the unit.

After the grit separator, the sewage flows down, under and up into the center of a sedimentation unit called a clarifier. In this unit the velocity is decreased so that 60 to 70 percent of the organic solids settle to the bottom where a scraper device then moves the solids to a collection pocket in the floor of the clarifier called the raw sludge hopper. As the sludge hopper fills with solids, it is necessary to pump out the raw sludge periodically to the digesters where it receives further treatment. The liquid portion of the sewage flows out of the clarifier over a peripheral v-notched weir plate to the effluent line which leads to an open ditch at the northwest corner of the site. The ditch carries the treated sewage in a northeasterly direction to the Prickly Pear Creek. Before leaving the treatment plant site, however, the sewage receives chlorination to kill harmful bacteria.


BYPASSED SEWAGE, STARTS AT FLOW OF
6.0.M.G.D & INCREASES PROPORTIONLY
AS THE TOTAL FLOW TO THE PLANT INCREASES
SLUDGE DRYING BEDS
PRIMARY DIGESTER
SECONDARY DIGESTER
CONTROL HOUSE
CLARIFIER
CLORINE LINES
GRIT SEPARATOR
INLET STRUCTURE
YORK ROAD

Raw sludge which has been pumped from the clarifier to the digesters consists of a mixture of water and organic solids. This mixture is oderiferous, highly putrescible, offensive and of a consistency of thin mud. It is extremely objectionable because treatment is difficult, and it cannot be dried on filter beds. Raw sludge generally consists of approximately 5 percent solids and 95 percenr water by weight.

Raw sludge is digested in enclosed tanks in an oxgyen-free environment in which anaerobic and facultative bacteria convert it. The raw sludge is broken into gases, dissolved salts and inert humus types of residue. The gases produced are combustible allowing them to be burned in a heat exchanger used to circulate and maintain digestion temperature.at 90 to 95°F. The process generally produces more gas than is required by the heat exchanger. When this occurs it is burned in a waste burner at a safe distance from the digesters. Most of the digestion takes place in the first, or primary, tank into which the raw sludge was pumped. After a given length of time, it is transferred to the second digester, which is actually a holding tank. Here the liquid separates from the humus and is drained off to the head end of the plant for retreatment. The digested sludge is more readily de-watered, having an average solids content of three to six percent by wieght. It is allowed to either flow by gravity or be pumped to the sludge drying beds north of the control building, where excess water drains off. The sludge then dries to a condition that enables it to be handled and disposed of.

Flow Records and Tests

The chapter on the sewer system pointed out that flow records have been kept from 1960 until the present time.

The average daily flow for the past year of operation was approximately 2.8 M.G.D. It is expected that this will increase to at least 4.5 M.G.D. by 1990 for the average daily flow and 6.2 M.G.D. maximum sewage flow. The peak flow that can be expected at the plant will have to include storm runoff from the combined part of the sewer system and will be between 20 and 25 M.G.D. Of this amount, 6.0 M.G.D. to 10.0 M.G.D. for a short duration can go through the plant. The remainder will have to bypass to be treated at the downstream end of the plant.

BOD, or Biochemical Oxygen Demand, tests and suspended solids tests have been performed on the incoming and outgoing sewage at the treatment plant once each week since 1965. Table 2 contains a summary of those tests.

Year	Average Flow During Tests	BOD Tests Average for Year Influent	BOD Tests Average for Year Effluent	BOD Tests Average for Year % Reduction	Suspended Solids Average for Year Influent	Suspended Solids Average for Year Effluent	Suspended Solids Average for Year % Reduction
1965 (last 5 months)	3.24	200.4	119.7	40.3	233.3	68.4	70.7
1966	3.3	247.9	140.1	43.5	237.5	72.3	69.6
1967	3.3	199.1	124.1	37.7	165.1	59.5	64.0
1968	3.2	199.7	129.7	35.1	170.1	56.4	66.8
1969	3.4	199.9	127.4	36.3	185.9	50.1	73.1
1970 (first 6 months)	3.57	187.0	109.4	41.5	194.3	59.8	69.2
Average for record period	3.33	205.7	125.1	39.2	197.7	61.1	68.9
Maximum for record period		523.0 (3-3-66)	254.0 (7-26-69)	68.0 (9-29-66)	660 (2-3-66) 553.0 (6-2-66) 525 (2-26-68)	113.0 (6-2-66)	92.5 (8-25-66)
Minimum for record period		109.0 (6-26-69)	55.0 (11-19-65)	3.4 (12-30-65)	100.0 (12 Diff. Times)	10.0 (6-26-69	16.7 (2-29-68)

BOD and suspended solids tests are an indication of the strength of the sewage. These tests on the incoming and outgoing sewage can be used to compare the degree of treatment being achieved with strength of sewage leaving the plant site.

The average BOD loading of the influent over the past five years was 206 parts per million (ppm) and the suspended solids were 198 ppm. During this five year period the effluent had an average BOD loading of 125 ppm and suspended solids of 61 ppm. The average reduction in the BOD was 39 percent and of the suspended solids a 69 percent reduction. The American Society of Civil Engineers Design Manual No. 36 states that primary treatment can be expected to remove 25 to 35 percent BOD and 50 to 60 percent of the suspended solids. This points out that the degree of treatment being given the sewage for the past five years has been better than normally expected. Except for rare occasions, there appears to be little variation in the loading or reduction of loading by the plant over that test period.

The average strength of sewage from the City of Helena is higher than the normal average that can usually be expected for a city of this size. Helena is about one-third stronger for BOD and ten percent stronger in suspended solids for comparable cities. This may be attributed to garbage grinders, which increase the BOD an average of 30 to 60 percent and suspended solids as much as 60 percent.

<u>Existing Problems and Recommended Solutions</u>

The existing plant has been and is presently giving excellent results in the reduction of sewage strength. However, there have been a number of reoccuring problems on the operation and maintenance that should be corrected to maintain this performance and lower costs. The following are the evident problems and the respective recommendations. (See Figure 3.10)

- Coarse grit arriving at the inlet structure has been a severe problem at times. It generally is the result of storm runoff washing it to the plant at peak flows when volocities are high. The source of the coarse grit appears to be from winter sanding operations and at other times seal or chips from pavement surface treatment projects. Coarse grit washing into the inlet structure against the grinding device causes the teeth on it to wear and break. There have also been times when grit has jammed the grinding device and caused shear pins to fail. To correct the situation it is recommended that a combination screening and coarse-grit removal piece of equipment be installed at the head end of the inlet structure with a design capacity of 6.0 M.G.D.

- The existing combination screening and grinding device has worn out and is giving a considerable amount of operating problems, especially in extreme sub-zero weather. These weather conditions tend to stiffen up the device and make shear pins fail. It is recommended that it be replaced with a new piece of combination screening and grinding equipment in which the grinder moves up and down across the screen.

- The inlet structure and grit separator are susceptible to many freezing problems during severe cold winters. It is recommended they be enclosed in a building.

- The existing mixer for the primary digester has failed. For best operation of the digester it is recommended that this be re placed immediately.

- The existing sludge gas meters in the control house are no longer working. Gas meters are necessary because gas production is an indication of the efficiency of sludge digestion. It is recommended that the gas meters be replaced with new ones.

- The drain hoses, insulation and roofing on both floating digester covers have failed. These should be replaced with new ones.

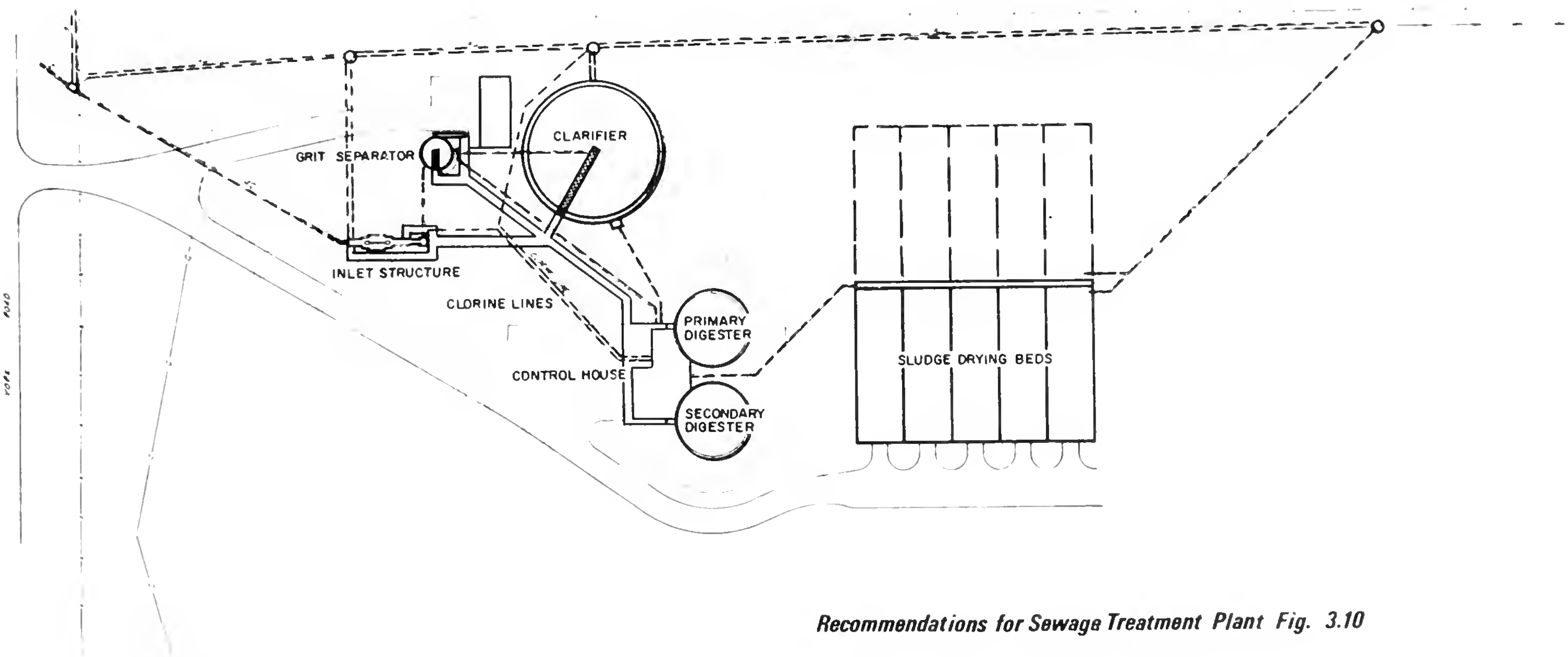
- Handling of the one ton chlorine containers is difficult, inconvenient and expensive. A new chlorination station should be built and the old chlorination room used for storage or a work shop. Costs of this are included with the recommendations for secondary treatment.

- There is no warehouse storage facilities for sewer cleaning equipment, tools, tractors, hoses, landscaping equipment, etc. It is recommended that a new combination garage-warehouse be constructed to enable equipment to be stored out of the weather.

- Additional water supply should be provided for the plant and especially before secondary treatment is installed. It could be an additional deep well or by bringing city water from the airport area, especially if the proposed new terminal building is constructed in the northwest part of the airport property.

Costs of Correcting Existing Deficiencies

The following cost estimates are submitted based on present day unit prices. They should be adjusted to reflect inflation at the time the improvements are made.



Recommendations for Sewage Treatment Plant Fig. 3.10

New trash rack & coarse grit removal unit	$ 22,000
New combination screening & grinding unit	22,500
New enclosure for inlet structure	9,360
New building for enclosure of existing grit removal unit	7,680
Digester repairs - new insulation and roofing for floating covers, drain hoses, gas meters, primary mixer	15,000
Storage garage & equipment building	9,600
Additional water supply (well & building)	5,000
Total Improvement Costs	$ 91,340

Capability of Existing Facility to Meet Present and Future Needs
The facilities at the existing sewage treatment plant are capable of providing acceptable primary treatment to Helena's average daily sewage flow until 1990. This includes growth within the present city limits in addition to potential annexation areas on the east and west sides. It is not capable of treating all combined storm and sanitary sewage flows in excess of 6.0 M.G.D. It also is not capable of handling the additional amount of sanitary sewage that might be pumped to the plant from the Helena Valley area.

The processing of diluted sewage that now bypasses the plant at flows in excess of 6.0 M.G.D. will require new equipment and facilities on the bypass. Meters to measure bypassed sewage may also be necessary.

To accept and treat the expected sewage contribution from the Helena Valley area would necessitate construction of additional sedimentation, grit removal and digester units.

The design of sewage treatment facilities requires the review and approval of governmental agencies. The Montana State Board of Health is the review and regulation agency on these matters. The Board enforces the present day water quality standards. The treatment provided by primary plants with 30 to 35 percent BOD removal is no longer acceptable. It is required that the degree of treatment remove 90 to 95 percent of the sewage strength, which necessitates that some type of secondary treatment, or its equivalent, be provided.

The existing digesters are currently operating at a theoretical capacity as determined by the American Society of Civil Engineers Sewage Treatment Plant Manual No. 36, pp 213. With a predicted increase in average daily flow to 4.5 M.G.D. in 1990 and an

installation of secondary treatment, the digesters will be too small. New units will have to be built or other means of sludge handling and disposal found.

SECONDARY SEWAGE TREATMENT

Primary treatment as previously described is mechanical in nature except for the digestion of the sludge, while secondary treatment is biological and involves utilizing aerobic or oxygen living bacteria. The two common types of mechanical secondary treatment being used are trickling filters and activated sludge. The activated sludge process is the more efficient, but more expensive to operate than activated sludge. On the basis of cost per pound of BOD removed they are about equal.[37]

Mechanical Methods of Secondary Treatment

Trickling Filters are one of the oldest methods of secondary treatment used. This involves applying thin sheets or spray of the primary clarifier effluent to a thick bed of coarse rock, stone or other type of filter media. The term "filter" is used to name the process but is a misnomer since little filtering takes place. The sewage flows down over the surfaces of the rock in the filter bed forming a gelatinous layer or film on the surface of the rock or other media. As the sewage flows form this film, aerobic bacteria among the filter media stabilizes the organic matter in the sewage. When the gelatinous layer grows thick enough, gravity causes it to slough off. The effluent, which is a mixture of treated sewage and sloughed-off gelatinous material, goes to a secondary clarifier for separation of the solid phase from the liquid. The sludge is generally pumped back to the head end of the primary clarifier, while the effluent flows on to chlorination and discharge to further treatment or a natural watercourse.

The degree of treatment usually obtained by this method is seldom over 90 percent removal of sewage strength. This type of treatment also requires a considerable amount of head loss, generally ten to fifteen feet. In this climate facilities should be covered to avoid freezing problems in the winter. Operating costs are relatively low, and it requires less attention while in operation.

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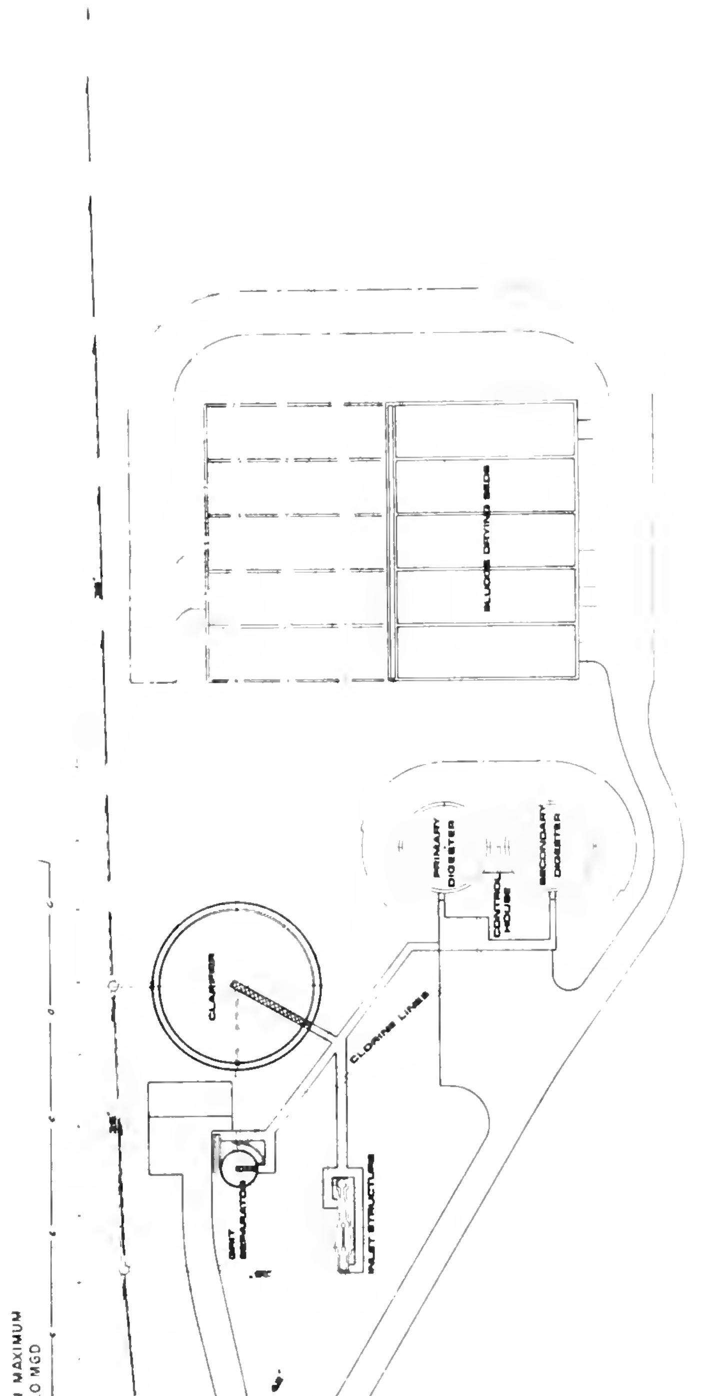
[37] Water and Sewage Works, November, 1968, Volume 115, No. 11, pp. 499 to 504.


BYPASSED SEWAGE, MAINTAIN MAXIMUM FLOW THROUGH PLANT TO 6.0 MGD
CLARIFIER
CLORINE LINES
GRIT SEPARATOR
INLET STRUCTURE
PRIMARY DIGESTER
CONTROL HOUSE
SECONDARY DIGESTER
SLUDGE DRYING BEDS
YORK ROAD

Activated Sludge is a development that came out of the use of trickling filters. It was found that when the previously described secondary sludge with its living slime organisms was recirculated and introduced simultaneously with an adequate supply of air at the head end of a secondary treatment unit, the degree of treatment was vastly improved. Another way of stating it is in the form of a biological process. The incoming sewage is mixed with the right amount of biologically active sludge. This mixture is vigorously agitated in the presence of air and final sedimentation permitted. The process is accomplished in large aeration tanks with the air being added by diffusion or mechanical means. The treated sewage liquor containing the activated sludge flows from the aeration tank to a unit for separating the solids phase from the liquid. This can be done by gravity in a conventional type clarifier or by the use of chemicals and air flotation whereby it can be floated off. The chemical flotation method gives a tighter sludge consisting of four to five percent solids and is more ideal.[38] Activated sludge from a clarifier can be as thin as 99.75 percent liquids. However, handling of it is a problem.

Activated sludge can also be handled quite conveniently in aerated lagoons by storing for a certain period of time, such as a year, and then disposed of by spreading over city-owned property at the lagoon to fertilize an acceptable agricultural growth.

Activated sludge does not settle out rapidly and in appearance is a light to dark brown flocculent material. It has a high water content varying from 98 to 99.75 percent and is difficult to de-water without chemicals. It is not odorous when fresh, but becomes septic if stored without aeration.

The activated sludge process provides a considerably better degree of treatment than the trickling filter and generally is expected to remove in excess of 90 percent of the sewage strength. However, it is a more complicated operation than trickling filters to maintain and operate.

The proper balance between returned sludge and raw sludge is important. The concentration of sludge in the aeration chambers must be kept within critically defined limits.

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[38] Water and Sewage Works, 1968 Reference Volume, Volume 115-RN, pp R178.

Biological Secondary Treatment

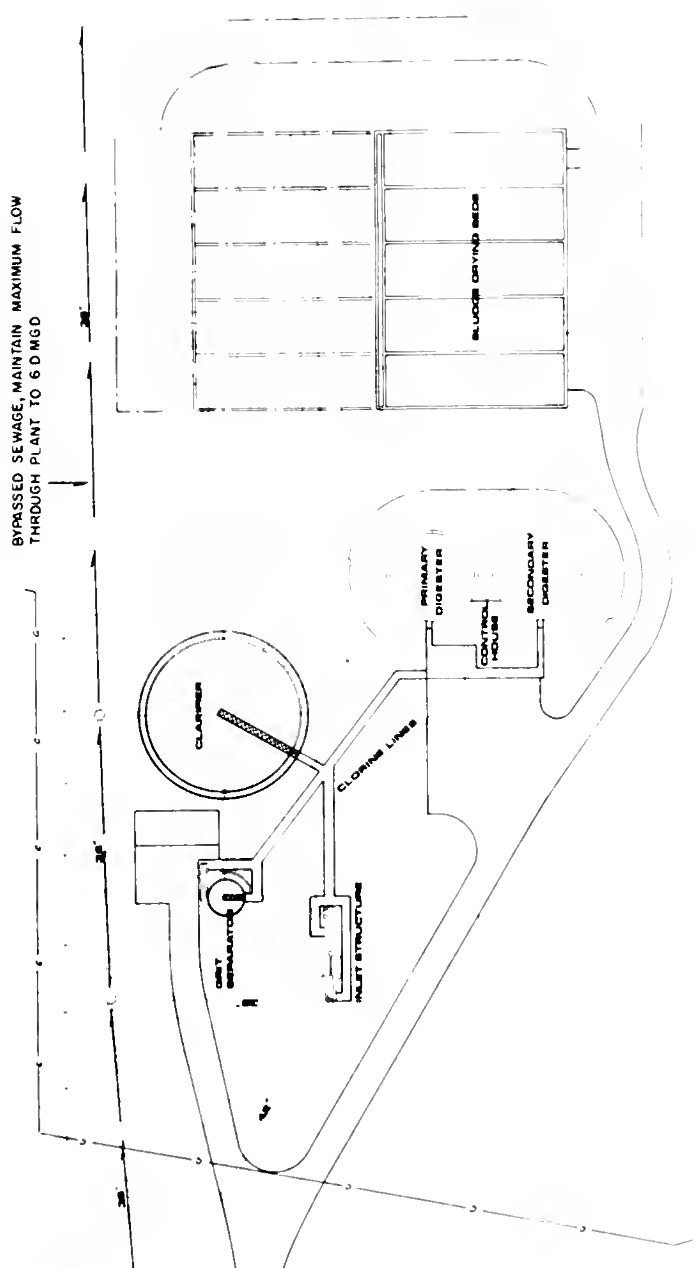
Waste Stabilization Ponds are shallow basins designed to treat raw, settled, intermediate or secondary treatment effluent by means of retention under conditions that favor a natural biological treatment accompanied with a bacterial reduction. It is a natural process that has been taking place in ponds, lakes and streams from the beginning of life on earth. It utilizes a complex relationship of bacterial and photosynthetic processes to treat organic contaminants in a pond.

Other names for this type of treatment are "oxidation ponds" or "sewage lagoons". The degree of treatment exceeds that provided by primary plants, and in many cases equals or betters that from secondary plants.

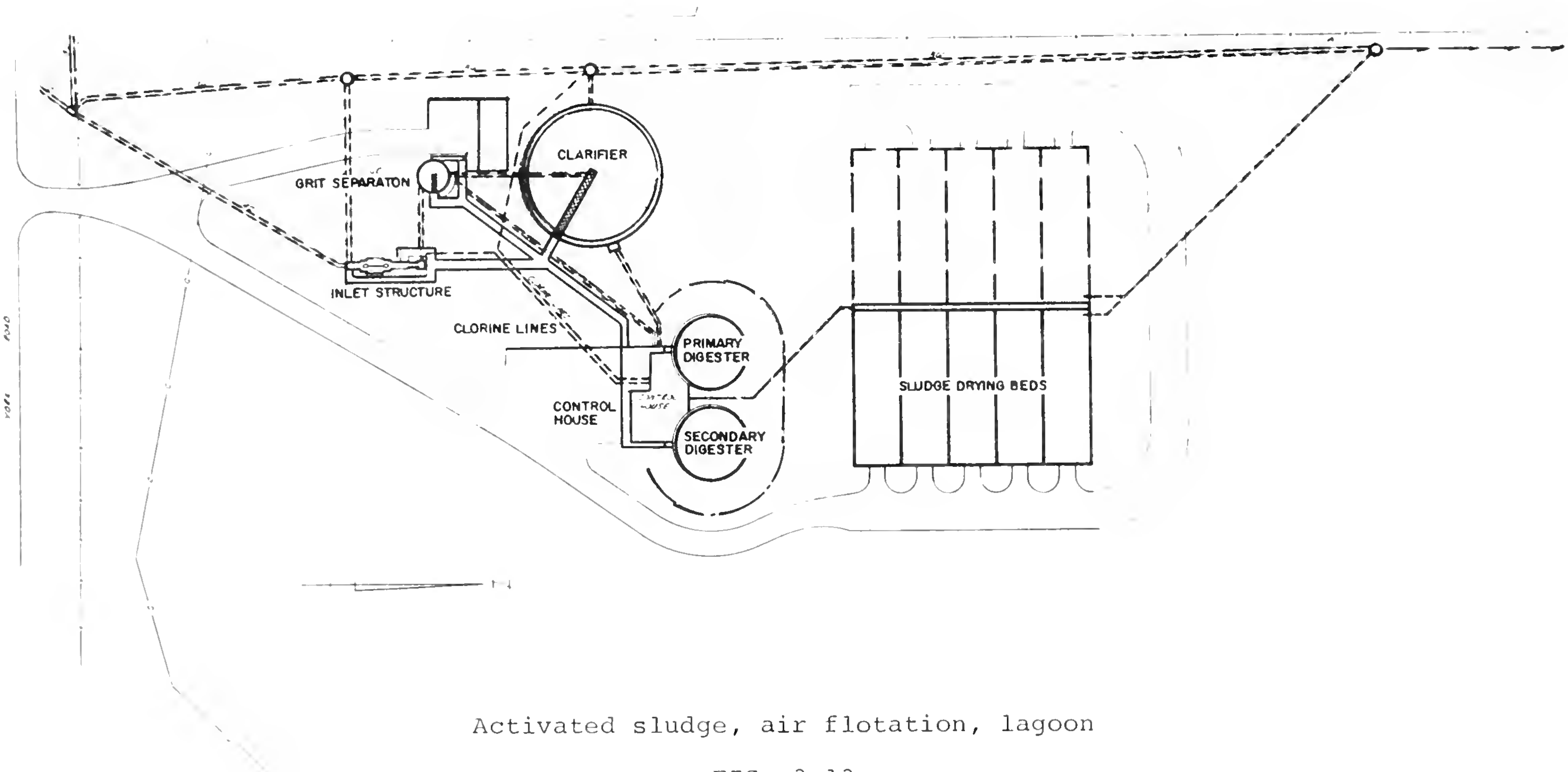
The requirements for design and construction of waste stabilization ponds are given in the "Ten State Standards for Sewage Works" published by the Great Lakes-Upper Mississippi River Board of State Sanitary Engineers. A few of the criteria are as follows:

- In terms of population, to have a minimum water surface area of one acre per 100 people for untreated sewage.
- In terms of BOD loading, not to have less than 1,000 square feet of water surface area per one-half pound of BOD loading.
- For natural aerobic functioning, ponds should be at least two feet deep but not over five, with a minimum freeboard of three feet.
- Provisions should be made for inlet, overflow and drain.
- Wood control and bank protection shall be planned for.

Land requirements for waste stabilization ponds are usually large and expensive, making their use for a city the size of Helena impractical. Approximately 325 to 350 acres of land would be required to construct a waste stabilization pond capable of treating the effluent from a primary plant with sewage flows the size of Helena's 1990 projected flow. The total cost of land and construction would be in excess of $1,250,000. The estimated size is not based on population requirements, but on BOD loadings as indicated by weekly sewage strength tests. The area mentioned above was determined by using a BOD application rate of one-half pound per 1,000 square feet of surface area per day, and a loading of 65 percent of the BOD of the raw sewage entering the plant.

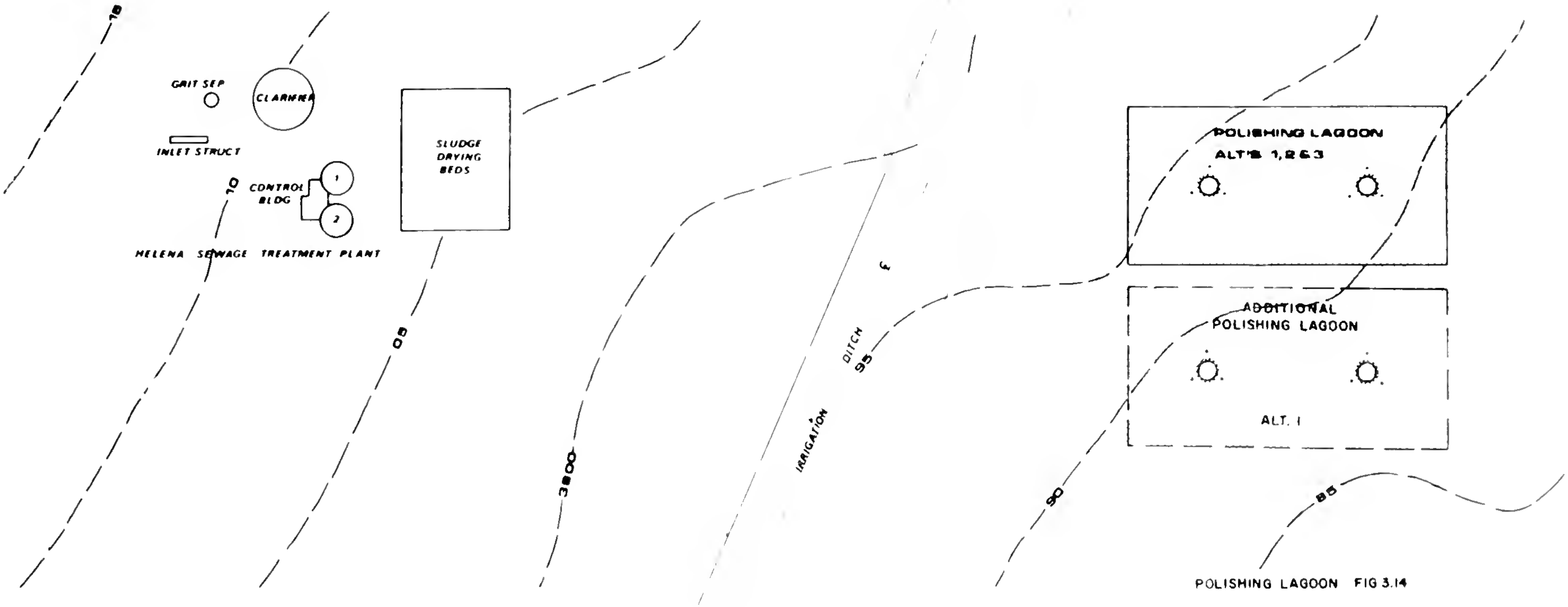


BYPASSED SEWAGE, MAINTAIN MAXIMUM FLOW
THROUGH PLANT TO 6 D MGD
SLUDGE DRYING BEDS
PRIMARY DIGESTER
CONTROL HOUSE
SECONDARY DIGESTER
CLARIFIER
CLORINE LINES
GRIT SEPARATOR
INLET STRUCTURE
YORK ROAD



Activated sludge, air flotation, lagoon

FIG. 3.13



POLISHING LAGOON FIG 3.14

In order to treat the effluent from a secondary treatment plant handling the same amount of sewage and capable of removing 90 percent of the BOD loading would require an oxidation pond having approximately 120 acres of water surface area at an estimated cost of $500,000. Additional expensive land would have to be purchased by the city to meet these requirements.

The only biological treatment method discussed so far has been waste stabilization ponds that utilize natural processes exclusively for treatment. The land area requirements can be reduced by modifying the depth, surface area and the addition of mechanical aeration.

The use of a small mechanically aerated stabilization pond on the effluent of the future secondary treatment plant should be considered for the following reasons:

- It can give final treatment to the remaining BOD in the plant effluent to provide a degree of treatment over 95 percent BOD removal.
- The diluted raw sewage bypassing the treatment plant during peak storm runoff will receive treatment as needed in this pond with only limited land requirements.
- In the event of failure or need for maintenance work on the existing clarifier, bypassed sewage would still receive treatment.

Sludge Disposal

Disposal of sludge is one of the major problems encountered in the design and operation of primary and secondary sewage treatment plants. The problems are due to the high water content, which results in a volume many times greater than that of its constituent solids.

To enable treatment of sludge in the simplest manner, it is necessary to reduce its volume and control its putrecibility. This can be done by reducing the water content as much as possible and altering the physical-chemical characteristics. The most common types of sludge treatment are:

- Thickening, an operation in which the sludge is slowly stirred for prolonged periods of time for the purpose of allowing a heavier and tighter sludge to form with less water content.
- Chemical conditioning, a chemical treatment used in improving the de-watering characteristics of the sludge.

- Digestion, a biological operation which utilizes the anaerobic (lack of air) decomposition of putrecible matter into gases, liquids and remaining solids (approximately 40 percent reduction of solids).

- Air Drying, an operation whereby sludge is allowed to drain and dry by evaporation on drying beds, usually sand.

- Vacuum Filtration, a mechanical operation that uses a vacuum or suction through a porous medium on the sludge to withdraw water.

- Incineration, an operation in which the sludge is dried completely, ignited and burned, alone or with auxiliary fuel.

- Sludge lagoons, natural or man-made earthen basisn used for the storage, digestion or de-watering of sludge. Aerated lagoons are especially convenient for handling high water content activated sludge.

Helena's present method of handling sludge is to reduce the volume and affinity for water of the raw sludge by using a pair of digesters to produce a digested sludge. The digested sludge is drawn at intervals to sand drying beds for de-watering and drying. Digester gas is used for heating the control building and the sludge in the digesters. When excess digester gas is produced, it is burned off in a waste burner at the site.

TERTIARY TREATMENT

Tertiary treatment is generally a combination of chemical and filtration methods of treatment applied to the effluent of secondary treatment plants. Its function is to remove nutrients that are in the form of nitrogen and phosphorous compounds. Primary and secondary treatment are not capable of nutrient removal. It is an expensive process that requires a great deal of justification before it is put into effect. The operating cost[39] alone is approximately 4½¢ per 1,000 gallons of sewage treated. For an average daily flow of 3.0 M.G.D. this would mean $135 a day, or $202 per day for the projected flow of 4.5 M.G.D. in 1990.

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[39] Technical Seminar Papers on "Nutrient Removal and Advanced Waste Treatment" held at Lloyd Center Auditorium in Portland, Oregon, February 5 and 6, 1969, pp. 5-13.

ALTERNATIVES FOR IMPROVING EXISTING SEWAGE TREATMENT

The deficiencies as outlined previously for the existing primary plant should be corrected at an approximate cost of $86,000. These are problems that need attention to maintain the present plant in good operating condition and are considered in the evaluation of updating the plant.

To update the plant to conform to present day requirements the following items will have to be satisfied:

- Provide treatment facilities for all bypassed sewage.
- Demolish and remove a portion of the existing sludge drying beds to make room for plant expansion.
- Provide a higher degree of treatment to conform to present day requirements, which shall at least be equal to or exceed secondary treatment.
- Improve sludge handling facilities.
- If the Helena Valley area sewage is to be pumped to the plant for treatment then increase the capacity of all primary treatment facilities.
- Install new chlorination, proportional feed facilities.
- Install metering facilities on the bypass and effluent lines.
- Provide additional water supply.

Considering the above mentioned requirements, alternate updating methods and cost estimates were prepared.

Regardless of the type of secondary treatment selected, the use of a mechanically aerated waste stabilization pond appears to be the most economical and simplest method to operate on the bypassed sewage. Peak flows occur that exceed 6.0 M.G.D. which produces bypassed sewage that will have to be given greater attention for treatment in the months of May, June, July and August. (See the summary of recorded flows, Table 3.) Occurrance of peak flows vary from one to six per year. This is not a high enough frequency to justify increasing the size of major units of the future sewage treatment plant, but the occurrance justifies treatment of the bypass flow, which presently flows on as diluted raw sewage to Prickly Pear Creek. Treatment used for the bypassed sewage can also be made to apply as the final polishing treatment on the

effluent of the plant. Another point in favor of a waste stabilization pond is that it will act as a safety factor for treating sewage that has, by necessity, been bypassed for repair or preventative maintenance of plant facilities.

When using a polishing pond to treat the effluent of a trickling filter plant, additional oxygen transfer and detention time is required as opposed to effluent from an activated sludge plant, which is more efficient in sewage strength removal. The final design of size and aeration capacity will depend upon: the average daily flow; and the average sewage strength in terms of BOD and suspended solids. Expected peak flow from storm runoff through the bypass facilities will also have to be considered. The duration of such a flow is seldom over four hours, and in most cases less than three.

Alternates considered in evaluating secondary treatment include:

- Trickling filters, secondary clarifier and polishing lagoon. (See Figure 11)
- Activated sludge, secondary clarifier and polishing lagoon. (See Figure 12)
- Activated sludge, air flotation of sludge and polishing lagoon. (See Figure 13)

TREATMENT PLANT IMPROVEMENT COST ESTIMATES

The following cost estimates are for the various alternates studied. They are based on present day unit prices and should be adjusted as needed for inflation. Sludge handling and disposal equipment are not included.

<u>Trickling filters, secondary clarifier and polishing lagoons.</u>

Two 100 ft. diameter trickling filters	$ 402,000
One 90 ft. diameter secondary clarifier	156,600
Chlorine contact chamber	50,400
New chlorination station	15,000
Two polishing lagoons with aerators	242,000
One sludge pumping station	20,000
Miscellaneous piping, etc.	20,000
Total	$ 906,000

Note: Because of the large amount of head loss that is needed to operate on gravity, the depths of sewage lagoons are great.

Consequently the total cost is more than would normally be expected. Filters are enclosed.

It can be expected that the annual operating cost will increase by $15,000 for adding this type treatment. By 1990 it will increase another $10,000 per year.

Activated sludge, standard secondary clarifier and polishing lagoon.

Aeration basin with mechanical aerators	$ 268,000
Secondary clarifier	132,000
Chlorination contact basin	48,000
Chlorination station	15,000
Sludge pumping station	20,000
Polishing lagoon with aerators	121,000
Miscellaneous piping, etc.	20,000
Total	$ 624,000

Activated sludge, air flotation of sludge and polishing lagoon.

Aeration basin with mechanical aerators	$ 268,000
Air flotation unit (enclosed)	237,000
Chlorination contact basin	48,000
Chlorination station	15,000
Polishing lagoon	121,000
Sludge pumping station	20,000
Miscellaneous piping, etc.	20,000
Total	$ 729,000

The annual operating cost of the treatment plant will increase by approximately $35,000 due to placing this type of treatment into operation. By 1990 it can be expected that it will increase another $15,000 per year.

The activated sludge, standard secondary clarifier and polishing lagoon proposal is the least expensive on an initial cost basis, but it is felt that the activated sludge, air flotation of sludge and polishing lagoon proposal should be given more serious consideration than the cost involved. It would be a more efficient operation presenting fewer problems with the collection of the activated sludge. In this alternate the sludge is maintained in an aerobic condition, which minimizes odor problems. This also satisfies the recommended procedure of rapid and aerobic removal of activated sludge to disposal. The buoyant nature of the sludge causes the blanket to rise above the surface of the water level, producing drainage of the water from the sludge and consequently

a higher solids content of four to six percent. A conventional clarifier normally produces a solids content from one-half to two percent.

ALTERNATIVES FOR IMPROVING SLUDGE REMOVAL

Three alternate methods for handling and disposal of sludge are proposed. They include:

- Expansion of the existing digester system and sludge drying beds.
- Use existing digesters for as much digesting as possible, but add sludge conditioning, concentration and de-watering equipment to supplement. Save half of the existing sludge beds for use and standby operation.
- Sludge reduction to a lesser volume of inert, sterile and inoffensive material by heat with or without chemicals.

Following are cost estimates for the three methods studied, which are based on average present day unit prices.

Expand existing digester system and sludge drying beds.

Two 45 ft. diameter digesters	$ 126,000
Sand drying beds	42,000
Building enclosure for drying beds to operate through the winter	64,000
Total	$ 232,000

Operating costs for the existing plant have not been broken down, but the total yearly operating costs for the plant operation and sanitary sewer systems runs approximately $65,000. Of this total it is estimated that $7,500 is needed for the digester and drying beds.

Use existing digesters to their capacity, but add sludge conditioning, concentration and de-watering equipment.

Two condition and de-watering units	$ 55,000
Two screw press, de-watering units	25,000
One building enclosure	7,500
Miscellaneous piping, connections, electrical, etc.	7,500
Total	$ 95,000

The operation of this additional equipment will increase the operating cost of the plant approximately $15,000 annually.

<u>Sludge incineration or other type of heat reaction to reduce sludge to a smaller quantity and easier material to handle.</u>

One plant installation (using Zimpro costs as a guide)	$ 275,000
One building	25,000
Miscellaneous piping, etc.	7,500
Total	$ 307,500

The annual operating cost would be approximately $15,000, but the volume reduction of the sludge would be 75 to 90 percent. The product would be a sterile, inert and inoffensive material.

RECOMMENDED SEWAGE TREATMENT PLAN

- On the basis of economics it is recommended that sewage treatment for the Helena Valley area be separate from the facilities for the City of Helena. The residents of the Helena Valley area must form either a Rural Improvement District or a County Sewer and Water District if they intend to have sewer facilities. This would require having one system for the Valley with one treatment plant, which in the long run will be more economical for all concerned. Land acquired for the treatment site should be at a location that would enable the entire district to be served by gravity flow to the plant.

- It is recommended that the existing problems and deficiencies for the primary treatment plant now in operation be corrected as outlined under the section entitled "Existing Problems and recommended solutions."

The approximate cost of making these improvements will be $91,300. If these improvements are made and the sewage from the Helena Valley area is treated in its own facility, the primary treatment units for Helena will be adequate until 1990. Other recommendations are as follows:

  - A polishing lagoon should be constructed on city property to the north of the Bureau of Reclamation canal. Its main function will be to catch and treat all bypassed sewage that occurs when the flow to the plant exceeds 6.0 M.G.D. Its secondary function is to give a final treatment to the plant effluent. The lagoon will have to be aerated in order

to conserve land space requirements, and this may be done with diffused or mechanical type aeration equipment. The design should be such that the rate of aeration can be varied to meet loading conditions and consequently save on electrical power costs for low aeration demands. The approximate cost is $121,000. The annual operating cost is estimated at $7,500.

- Secondary treatment facilities should be constructed on the effluent of the existing primary plant to upgrade the total degree of treatment now being provided for by the plant. Although it is more expensive and difficult to operate than trickling filter type plants, it is recommended that an activated sludge plant be constructed with a design flow of 6.0 M.G.D. to match the capacity of the existing primary plant and to meet the normal demands for the sewer system until 1990. Rather than a conventional gravity type clarifier, it is recommended that air flotation be used for separating the activated sludge, thereby giving a denser sludge easier to handle. To improve chlorine handling facilities and make room for a work shop, it is recommended that the present chlorination station be relocated to an independent new building on the site. A contact chamber should be constructed on the outfall leaving the plant. The approximate construction cost will be $608,000, and the annual operating cost is estimated to increase $27,500 over the present, with another $15,000 increase by 1990.

- The existing digesters and half of the drying beds should be kept and used to their capacity. To improve the sludge handling operations, it is recommended that two sludge conditioning and de-watering units be installed. They should have a capability of de-watering digested primary sludge, activated sludge or a mixture of the two. The construction cost of this equipment in place and ready for use is estimated at $95,000. The increase in the annual operating cost for the plant is approximately $15,000. There will be some savings in handling the sludge for disposal, but without cost records it is difficult to predict.

- Contact should be made with the Bureau of Reclamation to determine if the effluent from the treatment plant during the season that the Helena Valley Canal is flowing would be acceptable and the degree of treatment required. If the degree of treatment is adequate and acceptable, interest of the Bureau in buying or trading water rights for the effluent should be determined.

PRIORITY OF SEWER IMPROVEMENTS

The recommended priorities of improvements for the sanitary sewer system and its treatment facilities are given below. They have been determined on the basis of what will improve and correct the system the most from an immediate need point of view. It can be expected that private parties and groups of individuals may want to have the priorities changed to reflect particular situations which will alter the plan. This is only a guide and the final priority listing, including water system improvements, will require public discussion and review.

Sanitary Sewer System Priorities

- Within the city limits or close proximity:

  1st priority - construct relief sewers to prevent overloading during storm runoff in the depot to the treatment plant outfalls.

  2nd priority - construct outfall sewers to the areas of potential immediate growth and annexation as demand or political situations require.

- Outside the city limits in the Jurisdictional Area:

  1st priority - initiate a ground water control testing program in the Helena Valley.

  2nd priority - conduct educational programs for the people in the Helena Valley as to the need for sewer and water systems and promote the formation of an areawide county sewer/water district.

  3rd priority - the Helena Valley County Sewer and Water District should develop a more detailed plan with cost pictures on a proposed sewer system to present to the people in the area for their support.

Sewage Treatment Plant

- 1st priority - correct existing deficiencies in the present primary plant.
- 2nd priority - construct a mechanically aerated polishing lagoon that can temporarily act as secondary treatment until secondary treatment is constructed.
- 3rd priority - upgrade the present primary treatment plant to secondary treatment.

CHAPTER VIII

STORM SEWER SYSTEM

The topography south and southwest of Helena is mountainous with several draws flowing in a north and northeasterly direction. These eventually drain into Lake Helena. The major streams collecting runoff include Prickly Pear Creek, east of Helena, Ten Mile Creek, west and north of Helena, Seven Mile Creek, north of Helena, which drains into Ten Mile Creek, and Silver Creek, north of the Helena Valley. The latter drains directly into Lake Helena. These streams are shown on the Jurisdictional Area topographic map.

In general, the principal urban area east of Henderson Street eventually drains into Prickly Pear Creek, while the area west of Henderson Street drains into Ten Mile Creek. The slope south of the railroad tracks is sufficient to drain snow melt and storm runoff with practically no ponding. North of the railroad the slope is not as steep. Isolated areas are flat, which causes some ponding.

The natural drainage of the area is affected by the railroads and railroad spurs, the airport, dredge tailings north of the railroad tracks, irrigation ditches northwest of Helena and numerous roads and streets.

EXISTING STORM SEWER FACILITIES

An inventory of the storm sewer facilities was made from city records and limited field investigation. (See Figure 4.1) At the time of this report some storm sewer improvements were being made on Henderson Street and Montana Avenue. Some improvements were also being proposed for Last Chance Gulch. Available records are limited in many areas. Facilities include storm sewers, open ditches, connections to sanitary sewers and curb and gutters. Most surface runoff escapes to open ditches in the vicinity of the railroads and eventually drains into Prickly Pear Creek or Ten Mile Creek.

To evaluate the storm runoff in the greater urban area of Helena, it was necessary to delineate the principal storm runoff analysis zones. (See Figure 4.2) The acreage of these zones was computed and used to determine runoff as explained later on in this report. Following are the descriptions of major drainage areas affecting the existing storm drain systems of Helena.

Drainage Area Between Garfield and Linden

Other than culverts at street intersections and the new storm sewer on Henderson Street, there is no designed storm sewer in this area. The Henderson Street storm sewer crosses the railroad tracks and connects to an open ditch which eventually drains into Ten Mile Creek. The area immediately south of the railroad tracks has much flooding, which should be considered for future developments. Much of the runoff that collects along the tracks eventually percolates into the ground. Open ditches along Henderson Street should eventually be replaced with storm drains to reduce scour which occurs in these ditches on steep grades and subsequent silting in lower reaches at flat grades. Streets without curb and gutter are experiencing erosion, and there is some damage to private property. Eventual installation of curb and gutter will eliminate this problem.

Last Chance Gulch Drainage Area

This drainage area is served by a combination of storm drain systems and interconnections to the sanitary sewers. (See Figure 4.1) Generally storm drain systems terminate at Last Chance Gulch near the Armory. The flow then continues across the railroad tracks to the north. At the time of this study new storm sewer improvements are being proposed as part of Urban Renewal projects. A large number of storm drain inlets now connect to sanitary sewers, and many old commercial buildings have roof drains that also connect to the same sanitary sewers. The Urban Renewal project will correct and eliminate many of these interconnections.

A major problem that can be expected in the future is related to the runoff which collects at the dredge tailings north of the tracks. This runoff either percolates into the tailings or is ponded before excess runs off. With present development of the area covered by these dredge piles and future development between Villard Street and the golf course, drainage facilities in the form of a storm drain will have to be provided through the tailings, across Custer Avenue and to Ten Mile Creek. Presently only one 30" diameter culvert on Custer Avenue accommodates runoff that doesn't percolate or pond from the Last Chance Gulch drainage basin. The total area drainage is 8,520 acres.

Davis Street Drainage Area

In this area existing storm sewers start at Clancy Street and Rodney. They continue northeast past the junior high school across the railroad tracks at the Burlington-Northern Depot to an area at Cooke and Lamborn Streets on Phoenix Avenue. Here there are no storm sewers to accommodate the flows. It is all open ditched and grades are bad. Streets and other development

affect the natural drainage thereby causing ponding. To eliminate this, a storm sewer system should be constructed from Phoenix north a sufficient distance to provide drainage towards the northeast.

Capitol Drainage Area

Drainage from this area starts at Mount Ascension and runs in a direction slightly east of north to cross the railroad properties at the Burlington-Northern Depot. It accumulates at the same vicinity as for the Davis Street drainage area and consequently aggrevates the problems described in that area.

Helena's East Side

The surface runoff on Helena's east side collects from an area between Lamborn Street on the west to the interstate highway on the east, starting approximately at the south city limits and running northeasterly to the interstate highway drainage. Except for culverts crossing the highway, the runoff runs via surface.

Area East of Helena

This area lies between Helena and East Helena. Runoff arises from three sub drainages that cross the highway between the two cities. It flows north across the railroads and then is diverted immediately south of the airport runways, where it either percolates into the ground or flows to an open ditch at the east end of the runway. Development of much of this area will consist of commercial buildings having large roof areas and paved parking facilities. Resulting storm runoff will substantially increase and add to drainage problems that occasionally occur north of the railroad tracks and along the airport property. Planning for storm runoff facilities and implementation should be initiated before further development alters natural drainage now in the area.

Helena Valley Drainage

There is no designed storm sewers in the Helena Valley other than culverts at road crossings. Storm runoff and snow melt follow natural drainage channels and discharge to Ten Mile Creek, Silver Creek or irrigation ditches. During years of unusually high precipitation, overflowing of ditches and drainage courses cause limited flooding of some areas near Forestvale Cemetery and Ehlers Corner (Montana Avenue and Sierra Drive).

METHOD OF RUNOFF COMPUTATION

Computations for runoff were made using the widely known Rational Formula, Q = AIR. Q = discharge in cubic feet per second, A = drainage area in acres, I = rainfall intensity in inches per hour, and R is a runoff coefficient that is based on the nature of the average surface of the drainage area being considered.

Weather Bureau records indicate the following intensity-frequency rain storm data from the Helena Airport as:

2 years	-	0.5"/hour
5 years	-	0.8"/hour
10 years	-	1.0"/hour
25 years	-	1.2"/hour
50 years	-	1.4"/hour
100 years	-	1.6"/hour

The rainfall intensity in the mountainous terrain at and south of the city can be expected to be greater than the values shown because adiabatic expansion of the rising air over the mountains causes a decrease in the temperature of the air and consequently a release of a certain amount of what is known as oragraphic precipitation. However, since there is no recorded data available to justify the value selected, the amount shown at the airport was used. The frequency of the storm was selected as ten years, which gives a rainfall intensity of 1.0 inches per hour. Areas were computed in acres using a planimeter. Coefficients of runoff varied with the slope and nature of the drainage area being studied.

Flow records of combined sewage flow at the sewage treatment plant show that the rate of surface runoff from snow melt is somewhat less than rain storm runoff.

Drainage basins, existing storm sewers and runoff flows for the Jurisdictional Area are shown on Figure 4.2 and also are tabulated below:

Area	Flow (cfs)
A	47
B	55
C	72
D	392
E	198
F	94
G	16
H	72
I	27
J	21
K	17
M	110

Rational Formula

STORM SEWER RECOMMENDATIONS

Recommendations on the basis of very limited flow records of the various drainage basins in the Helena area can only be speculative. There is a substantial variation of precipitation intensities within the area studied but no records are available to substantiate the amounts. Snow melt runoff could pose more serious problems in ponding areas than rain storm runoff because much of rain runoff percolates into the ground; whereas on frozen ground it wouldn't seep in so fast. Efforts should be made to install storm drains in all areas where ponding exists, particularly the dredge piles area between the railroads and Custer Avenue and the area north of Phoenix Avenue from Montana Avenue to the interstate.

The use of open ditches in residential and commercial districts should be discontinued because of scouring at high flows and accumulation of unsightly debris along the ditches. The city should continue to separate the combined sanitary-storm sewer connections whenever practically possible and should attempt to bring the storm records up to date with field averages.

Development of any area tends to restrict or divert drainage from its natural course. Prior to such development, the City Engineer's Office should have the authority by ordinance to permit or deny any change in natural drainage. Many localized drainage problems have occurred because there was no planning for future development.

Urbanization will increase the amount of runoff, decrease the time of collection and alter natural drainage courses. Implementation of proper gauging and measuring devices will aid the design of future storm drain facilities and a realistic evaluation of the present facilities could be accurately made.

In many cities outside Montana the U.S. Geological Survey, Water Resources Section, has made surface runoff studies in cooperation with the city government. Financial assistance and technical know-how are provided on a sharing basis. Helena should probably engage in such a program.

In any areas where it is felt that it would be beneficial to separate the connections of storm drains or roof drains from sanitary sewers, but where there are no records of such connections, it is recommended that the use of some type of detection device such as smoke or dye be used to determine the amount of interconnections. These same methods might also be used in tracing lost storm drains not shown on the records in the city files.

APPENDIX A

METHODS OF DATA COLLECTION

A number of methods were used in studying the Helena water system to obtain the available information relating to the physical and other characteristics of water collection, transmission, treatment and distribution facilities. These methods included gathering all available maps, plans and reports from the city offices and other private and public agencies, conducting personal interviews with individuals and public agencies and by actual field review and measurement of visible water works facilities. Personal interviews were conducted with the following individuals and agencies:

- Water Resources Section of U.S.G.S. - Grant Buswell, District Engineer, Surface Water Division, Montana and Don Coffin, Ground Engineer, Groundwater Division, U.S.G.S.
- State Hydrogeologist, Maxwell Botz, Montana Bureau of Mines and Geology.
- Montana Fire Rating Bureau, Guy Crow
- Montana Board of Health, Mr. Art Clarkson and Don Willems.
- State Water Resources Board, Hans Dille, Surface Water Division
- Federal Housing Authority.
- City Fire Chief, Chief Brockway.
- City Commissioner, Ed Loranz.
- Farmers Home Administration, Ted Hebness.
- Montana Power Company, Missoula Water, Frank Head, Supervisor.

A number of reports and plans were available regarding Helena's water system. These documents have been studied and credit for information from these sources is given in the bibliography.

A review of all portions of Helena's water systems was conducted in company with the City Engineer and Water Superintendent. In addition, a field survey was made to look at Travis Basin to evaluate its use as a potential reservoir site for Helena. An inventory was made on Red Mountain Canal to measure and record lengths of varying type construction - flume on trestle, lined canal and unlined canal. A visual field investigation was conducted along Helena's wooden water transmission mains to determine visible evidence of leakage. Quantitative evaluation of leakage was conducted in cooperation with the City Engineer to determine the magnitude of such leakage by field tests with special flow measuring equipment pitometers and differential monometers). Considerable field investigation and a review of as-built construction plans were conducted in studying Helena's water treatment plant. Other field work was necessary to verify recorded data or to obtain that data not available from the records.

Records for reporting the origin of Helena's various water systems are very limited, but some information was obtained from the memories of Mr. George Malben, retired Helena Water Superintendent, and Mr. Herb Foote, who worked for many years with the State Health Department.

APPENDIX B

COMPUTER ANALYSIS OF WATER DISTRIBUTION SYSTEM

SOURCES OF INFORMATION

1. Water system maps (Helena City Engineer's Office and Montana Fire Rating Bureau).

2. Existing demands and consumption rates (Helena Water Accounting Office records).

3. Fire flow data (Helena Fire Department and Montana Fire Rating Bureau).

4. Pumping plant data (Helena City Engineer's Office and Helena Water Plant Superintendent).

METHOD OF ANALYSIS

A total of 63 different combinations of the proposed improvements to the distribution system and flow conditions were analyzed by computing the line pressures at 103 pipe junctions and flow rates in 168 pipes of a network containing the principal members of the Helena water system. The calculations were carried out on the digital computer and Montana State University using a program developed for the analysis of water distribution systems.

A brief description of the flow rates, the proposed improvements and the combinations of these which were used in the analysis are presented in the following paragraphs.

The flow demands used in the analysis included the following:

- A 35 million gallon per day (M.G.D.) demand for the maximum peak hourly demand rate based on a total projected population of 32,000 in the water system service area of 1990, including those areas projected outside the present city limits, applying an average consumption of 320 gallons per capita per day (gpcd). The maximum average daily rate is 2.2 times the average. The maximum hourly demand is 1.5 times the maximum average daily rate.

- The potential growth area impact demands included in this are 1.2 M.G.D. at Hiawatha and Cannon, 1.2 M.G.D.

on north Benton at the Burlington-Northern railroad tracks, 2.8 M.G.D. at Montana Avenue and Walnut Street, 1.3 M.G.D. at the airport, 1.3 M.G.D. at Winne and Roberts, 2.5 M.G.D. at Prospect and 19th Street, the new Montana Highway Department complex, and 2.1 M.G.D. at 19th Avenue and Fee Street. The balance of the 35 M.G.D. is distributed throughout the system with moderately heavy demands in the Central Business District and in the vicinity of the Capitol complex.

- A 16 M.G.D. industrial and domestic demand developed with similar considerations was maintained with the fire flows recommended for each of several locations in the city by the Montana Fire Rating Bureau. The fire chief of Helena indicated that there had been no major fires occurring simultaneously in the city. For this reason it was decided to use the 16 M.G.D. plus one fire flow at a given location for the analysis. Each proposed improvement was analyzed for nine or ten different fire flow conditions.

- A 17.3 M.G.D. industrial and domestic demand was used to determine the effectiveness of the proposed improvements for supplying the city when the pumping plant was not operating.

Six proposed alternate improvements to the distribution system consisting of four specific piping changes or combinations thereof were considered. The specific proposed improvements are shown on the system map (Figure 2.7) and are described by references in the body of the report.

- South Side: A 16" diameter line to be installed on 3rd Street from Dakota Avenue to Davis, on Davis from 3rd to State, and on State from Davis to Last Chance Gulch. A 20" diameter pipe to be installed from the intersection of State and Last Chance Gulch running in a northwesterly direction to a connection with the 24" line leading from the west end of Holter Street to the Woolston Reservoir. The connection to be located midway between Holter and the reservoir.

- Cross-town on Broadway: An 18" diameter line on Dakota Avenue from 3rd Street to Broadway to give a combined capacity with the existing 12" line equivalent to that of a 20" line. A 16" diameter line to be installed on Broadway from Dakota Avenue to Last Chance Gulch, on Last Chance Gulch to Edwards, and on Edwards to Benton*. A 20" line to be installed

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*Note. This location is in conflict with the Urban Renewal Plan for streets. Adjustments were made to conform to on-site street improvements planned by Urban Renewal. (See Water System Plan, Figure 2.8)

from Benton and Edwards running northwesterly to the connection with the 24" line between Holter Street and Woolston Reservoir.

- West Side: A 10" diameter line to be installed on Benton from Hauser Street north to Meadow Street, west along Meadow Street to Brady Drive. This segment is referred to by pipe numbers 12, 15 and 23. A 16" diameter line to be installed on Laurel from a connection with the 24" and 16" transmission lines from Ten Mile Reservoir to Hauser Street. This segment is referred to by pipe numbers 13 and 14.

- East Side: A 16" diameter line to be installed on Walnut Street from Montana Ave nue to connect with the existing 20" line terminating at the southwest corner of the Helena Municipal Airport. A 12" diameter line to be installed to connect the intersection of Roberts Avenue and Walnut Street with the existing 12" line terminating at the depot on the south side of the Burlington-Northern railroad tracks. A 10" diameter line to be installed along Montana Avenue from Boulder Street north to the Burlington-Northern railroad tracks.

Combinations of these improvements, tabulated on the water system summary sheets, were analyzed for different flow conditions according to the following groupings:

- Conditions 1-7: The performance of the existing system was compared with the six proposed alternate improvement plans for a projected maximum hourly demand rate of 35 M.G.D. The proposed alternate plan using the South Side line was eliminated from consideration at this point. This reduced the number of alternatives to four, all of which included the cross-town connection on Broadway.

- Condition 8-55: The performance of the four remaining alternates were compared using a projected daily demand rate of 16 M.G.D. plus maintaining a fire flow at one location at a time in the system. The effect of nine or ten fire flows were examined for each alternate. This analysis eliminated the 16" line on Laurel from consideration but showed a definite need for increasing the capacity of the line on Benton to provide adequate fire protection in the vicinity of Carroll College. The results of this part of the analysis are tabulated on pages 2, 3, 4, 5 and 6 of the water distribution summary sheets.

The conditions 1-55 indicated that the conditions of peak hourly demand rate and the condition of 16 M.G.D. plus fire flows may be met without implementing the proposed East Side improvement alternate.

- <u>Conditions 56-59</u>: The effectiveness of operating the pumping plant during the hours of minimum demand in the peak water use season as a means of increasing the rate of refilling Woolston Reservoir was examined using four alternates: the cross-town line on Broadway with and without the proposed East Side improvements when flow is going into the east side reservoir at Alta and Seminole and into Woolston Reservoir, and with and without the proposed East Side improvements when flow is going into only the Woolston Reservoir. (The East Side reservoir is valved off by installing an altitude control valve.) This analysis showed the altitude control valve in the East Side reservoir line to be much more effective than the proposed East Side improvement as a means of increasing the rate of refilling Woolston Reservoir. The results of this part of the analysis are tabulated on page 7 of the water distribution summary sheets.

- <u>Conditions 60-63</u>: The effectiveness of four of the proposed improvement plans were compared for a flow condition of 17.3 M.G.D. average daily demand when the pumping plant is not operating. The cross-town connection on Broadway is common to all four plans. This was run in combination with the following improvements: both east and west side improvements together, west side improvement, increasing the capacity on Benton, and east side improvement. The results of this analysis showed that the effectiveness of the system was not appreciably altered by adding the east or the west side improvements. The addition of the improvement on Broadway only will permit the system to carry a 17.3 M.G.D. demand. The results of this part of the analysis are tabulated on page 8 of the water distribution summary sheets.

RESULTS OF COMPUTER ANALYSIS

A partial listing of the results of the computer analysis are presented in pages 1 - 8 of the summary sheets for the water distribution system. The summary gives the line pressures at the 17 most critical junctions in the system for all flow conditions except numbers 56-59 inclusive. For the latter, the summary shows only the reservoir and pump station flows.

The flows provided by the pumps and the reservoirs are also tabulated for each flow condition.

The junction numbers associated with the pumps and reservoirs in the summary sheets indicate junctions where flow enters or leaves the distribution system going to or from the reservoirs. The interconnection just north of the Woolston Reservoir causes an equalization of the pressures in that area permitting the three single lines leading from the junction numbered 37.45.46 (see Figure 2.7) to act as if they terminated in a single reservoir.

The complete listing of pressures at all 103 junctions for all 63 flow conditions are on file with Northwest Planner, Helena; the Helena City Engineer; or William A. Hunt, P.C., Bozeman, Montana

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